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Purchasing Coal by Specification.

It is gratifying to learn that good progress is being made by the United States Government in purchasing coal according to its value as determined in the laboratories of the Geological Survey. When the Government began to buy coal in this manner, great opposition was encountered, so much so that in some cities it was practically out of the question to secure any bids under specifications calling for deliveries agreeing with samples submitted by the bidders and paid for in accordance with the heat-unit and other properties of the fuel actually delivered. Experience has shown that after dealers once become acquainted with this method of making sales their fear of it passes away, and occasionally one of them is frank enough to say that the method is actually of help to them in furnishing information concerning the products they are handling and in the degree of uniformity secured by it. The method is really a great advantage to the Government, as the latter maintains public buildings all over the country and pays out an enormous sum annually for coal bills. In fact, probably no single interest purchases fuel over such a wide extent of territory, and consequently the investigations being conducted to determine the value of coal in all parts of the country are of specially direct benefit to the Government as a large purchaser. For this reason alone the research work done by the fuel-testing laboratory is fully warranted, while the fact that all of the results obtained are made public, so they can be obtained by anyone adds greatly to their importance. The Government is now buying 400,000 tons for use on the Isthmus of Panama alone, which is some indication of the extent of its purchases.

While the primary purpose of these investigations is to collect representative samples from the various coal fields of the country and to analyze and test these samples so the quality of the various grades of coal may be definitely known, other results of the work are almost as important. At the present time an enormous amount of coal is practically wasted, for one reason or another, between the mines and the cinder pits. The Government engineers and chemists are making a careful study of the nature and extent of all these wastes and already have furnished interesting information on the subject. In connection with this work attention is being paid to methods of utilizing low-grade fuel, such as lignite, peat, slack, washery refuse, and culm. In some sections of the country there is an abundance of these low-grade fuels, but they have not been employed to any extent, except in an unsatisfactory and very wasteful manner, and fuel has been brought into these places at high expense for freight. There are good reasons for believing that for many purposes some of these low-grade fuels can be employed satisfactorily, at a material reduction below the present prevailing prices, for generating steam or heating with fuel from distant fields. A secondary feature of the investigation which is attracting considerable attention is the success attending the endeavors to abate smoke. This work is the endeavor to obtain increased efficiency when burning coal, but it appeals personally to the people of so many communities that it probably is attracting more attention than the technical research of greater actual importance.

A feature of the investigation that is being followed with considerable interest by many users of small power plants relates to the utilization of low-grade fuels in producers. The manufacturers of suction-producer plants have been paying much attention to this subject and have accomplished creditable results in many cases. Unfortunately, however, for the rapid progress in the art, plants of this sort have been entrusted frequently to the care of men who fail to appreciate the importance of looking after them properly. As a result, while the producers have generally operated satisfactorily, the gas from them has often proved hard on the engines and the operation of the latter has been unsatisfactory. The explanation for this is that the engines have not been properly cleaned, a fact that the owners have generally been unwilling to admit, or are not sufficiently educated technically to recognize. Their troubles have been ascribed by them to defective equipments, which may have been the case in a few instances, but probably would not hold in the majority. These owners have hurt the progress of small producer plants in popular favor by statements concerning their own unfortunate experience. The assertions of the manufacturers of such apparatus have accordingly, in many cases, been attributed to an unwarranted belief in what can be done by such plants. The fuel-testing laboratories of the Government have shown, however, by many tests under actual working conditions that low-grade fuel from many different parts of the country can be employed in producers very satisfactorily without causing injury to the engines using the gas. These tests made by the Government are absolutely impartial, and the fact that those directing them conclude that the producer is actually better than a steam boiler for utilizing some of these fuels should go a long way toward convincing the public of the merits of small gas-producer plants when looked after with the same care as steam plants.

A great deal still remains to be learned concerning these small suction-producers, particularly when working on low-grade fuels, and the management of engines operating on such gas is not properly understood by those who have used only a gas that is fairly free from tarry matters. As the work of the Government and private parties furnishes additional information it is reasonable to expect that the present prejudice against small plants will disappear, and a new and extensive use will be found for some classes of fuel that are at present ranked as hardly worth using. The demand for good coals is now so extensive and growing so rapidly that anything which will increase our knowledge of the utilization of the poorer grades deserves hearty support. While the Government, as a purchaser of fuel, gains directly by these investigations, the gain to the public is so great that the work should receive general encouragement.

The Preliminary Treatment of Clay-Bearing River Waters.

In purifying water in America there have been two different and widely separated types of apparatus. One of these is the sand filter, in which the passage of water through sand, at a comparatively low rate, has been mainly relied on for the purification; the other is the so-called mechanical filter, in which the chemical treatment of the water prior to filtration has been the main reliance, this treatment serving to change the nature of the finely-divided particles in the water so as to bring them together into aggregates in order that they can be easily and completely removed at a higher rate and through a coarser filtering material. In the last twenty years a large part of the discussion of filtration has been directed to the relative merits of these two general types, and of their respective availability under various conditions.

At the beginning sand filters only were considered. Filters of this type had been developed in Europe, where they had been widely used in treating river waters for municipal use, with results that had generally been favorable. The first plants built for the purification of American waters followed European precedent, dependence for purification being placed mainly on the slow passage of the water through the sand. Before the application of this process had gone very far, however, it was found by practical experience that the clay-bearing waters of many western and southern streams could not be adequately purified in this way.

What was known in a practical way by a number of people at a comparatively early date regarding the purification of such waters was explained and made certain by the classical experiments of Mr. George W. Fuller, at Louisville, in 1895-6, and at Cincinnati, in 1898. Since the publication of the results of these experiments there has been no question of the general fact that methods of purification previously used in treating European waters, and some of the waters of northern and eastern States of this country, which were not clay-bearing, could not be applied to the clay-bearing waters of the West and South. On the other hand, there has been a strong, wide-spread and persistent prejudice against the chemical treatment of water. It is called a prejudice, because there is no real reason to believe that water which has had a proper chemical treatment is less wholesome in any way than water that has not been so treated.

The recognition of the fact that sand filters, following European practice as then developed, were incapable of treating many waters that required purification, led to numerous and persistent efforts to so modify, amend or correct the process as to make it applicable to those waters. In these efforts the effect of the prejudice against chemical treatment had much to do. Mr. Fuller found at Cincinnati that water which carried only a little clay could be successfully purified by sand filtration, while with a larger amount satisfactory results could not be obtained. Consideration was then given to the removal of some of the clay before sending the water to the filter, with the idea of reducing the amount of clay within the limits which Mr. Fuller had determined could be taken care of by the latter.

This certainly seemed like a promising proposition, and it has been followed consistently. Scrubbers and preliminary filters have been advocated, experimented with and built at great expense to remove some of the suspended matter from waters that were afterward to be passed through sand filters. Unfortunately the desired results have not followed this treatment, for the very simple reason that the matters which are removed by the scrubbers and preliminary filters are the coarse-grained part of the turbidity, forming a part of the matters which would be removed by the filter without question in any case, while the finest-grained particles, which the filter is unable to remove, are not stopped in any appreciable degree by the scrubbers and preliminary filters. It is true that the whole amount of sediment is reduced, but the troublesome part of it is not reduced in proportion. The limits which Mr. Fuller found for the amount of turbidity that could be removed by his experimental sand filters are those that could be applied only to the particular conditions under which the experiments were made. For that reason, therefore, they are not general limits.

The question as to whether a sufficient purification of a clay-bearing water can be obtained by sand filters depends more on the character than on the amount of the turbidity. To put this more accurately, perhaps, the success of the process depends on whether or not there is a considerable quantity of suspended matter of such fine-grained size that it is capable of passing through all the scrubbers and preliminary filters and final sand filters that may be put in its way. This fine-grained turbidity is easily and completely removed by chemical treatment, because the chemical serves to coagulate it. In other words, it draws the separated minute particles into groups or aggregates, which, because of their greater size, are easily and completely removed by filtration.

The project for the Washington filters, as recommended in 1901, was novel in that it combined some of the features of both systems of filtration which had previously been discussed and used. It was proposed to build sand filters, since in that way the greater certainty of purification, as compared with results from the mechanical filters which had been used in this country up to that time, was to be obtained. During at least eleven months of the year, when the Potomac water is not very turbid, the results obtained by the sand filters would also be sufficient. During these eleven months the very considerable expense of coagulant, necessary with the other system of filtration, would be saved. During the other month, when the Potomac water was incapable of adequate purification by sand filters alone, the water was to be given a chemical treatment. In that way, and only in that way, could full purification be obtained at all times. The project was adopted in part as the sand filters were built, but the chemical treatment, which was a necessary part of the plans, was excluded because of the prejudice against the use of chemicals in the water.

The actual experience with sand filtration of this water has shown the insufficiency of treatment during the periods of high turbidity of the Potomac. Experiments, conducted on an adequate, thorough-going scale, show beyond a doubt that preliminary filters or scrubbers would not materially improve the conditions in this respect. These experiments are admirably described and discussed elsewhere in this issue by Mr. Francis F. Longley, under whose direction they were made. Mr. Longley's experimental results are entirely in accord with experience in actual use where preliminary filters have been built for the treatment of clay-bearing waters. They also seem absolutely conclusive as to the efficiency of these devices as applied to the Potomac water.

Mr. Longley generalizes, and says that the evidence tends to show, and that there is every reason to believe, that this removal of clay particles cannot be effected by any device applicable to large quantities of water, which involves purely mechanical principles in its operation; or, to put it in another way, he concludes that no system of scrubbers or preliminary filters, however perfect, is capable of correcting the acknowledged defect in the plant. It is dangerous to generalize, but in this case the generalization seems well grounded, especially when the additional experience of plants with preliminary or double filtration which have been actually used in treating clay-bearing waters is taken into account, and where, in some cases, turbid effluents have been produced, and in others where chemical treatment has been resorted to as an auxiliary at times of exceptionally turbid water.

Mr. Longley's experiments on double and triple filtration are especially interesting, and seem most conclusive as to the extraordinary degree of subdivision of this finest turbidity and of its persistence in passing through the sand of one filter after another. Scrubbers and preliminary filters may have their place in treating waters of certain kinds, but from the results of these experiments and other experience, their use clearly does not extend to the treatment of clay-bearing waters. The completion of the Washington project has been delayed for some years to allow these matters to be investigated and to allow prejudice to be satisfied. The results obtained are certainly conclusive, so it is to be hoped the works may be completed without further delay, and that Washington may at last have the thoroughly purified water to which that city is entitled, and for which substantially all the money necessary has been spent.

Interurban Railway Investigations.

Whenever it is proposed to build a new steam railroad through a territory already more or less satisfactorily served by transportation facilities, an exhaustive study of past and present conditions with conservative estimates of future developments are clear necessities on the part of the projectors of the enterprise. The more highly developed the State in which the project seeks the right to live, the more essential it is for those behind the new plans to be able to show that the facilities are not purely destructive of existing service, but are desirable additions or supplements to it. In other words, the tendency of the times is away from unrestricted competition and favorable to the operation of so-called monopolies under public supervision and control. This does not mean that competition is not tolerated in a highly developed State, but it does mean that every new enterprise which rivals others previously in the field must show just cause for its establishment, lest the public be forced to pay for a dual and perhaps unnecessary service. The burden of proof is fixed upon the promoters of the new project, and the demonstration is largely an engineering problem.

The development of electric traction during the past ten years has followed the lines of connecting cities and towns heretofore unprovided with interurban trolley service. As speeds and car weights have increased, the standards of operation and comfort have been raised, until in many instances the conduct of an interurban road is as responsible a task as the administration of a moderate-size steam railroad property. While the operation of such a road by electricity is different in many details from the conduct of a steam railroad, the character of the service given demands no less thorough precaution against accident, no less complete knowledge of handling traffic efficiently, and no less clear a definition of legal rights and restrictions than obtain in the case of the road using the older motive power. The high-speed electric railroad can come into its legitimate field of usefulness only after it has demonstrated its standing on practically the same plane as its steam-operated competitors, and this means the presentation of engineering testimony which is often of great analytical interest.

A case of this character is now pending before the Massachusetts Railroad Commission in the project of the Boston & Eastern Electric Railroad Company to build a high-speed electric interurban line between Post Office Square, Boston, and the northern suburban district bounded by Salem and Beverly. The company desires a certificate of public convenience and necessity from the commission which will enable it to secure its locations in the various cities and towns traversed, and at hearings extending through the past year and a half it has been endeavoring to show the need of its construction and operation. However the board may decide as to the certificate, the thoroughness with which the engineering problems and testimony have been prepared renders the methods followed in the investigation of wide engineering interest, for few electric railway projects in the East have had to face stronger opposition from existing transportation companies. The testimony already taken before the commission covers more than 1500 pages of typewritten copy, including direct and rebuttal evidence, cross-examinations and arguments. It is estimated that the preparation of the case so far has cost the company some $20,000. Among the points that have received special consideration from the engineering standpoint are the estimated cost of the road in detail, rather than in the rough, on a per mile basis; the growth of population and existing train and trolley service in the territory on competitive lines for a number of years back; estimates of present and probable future traffic and earnings derived from reports, personal tests and counts of the present facilities, coupled with studies of the per capita and per mile earnings of all the principal high-speed electric interurban roads of the country; studies of the types of rolling stock best adapted to rapid loading and unloading of passengers; detailed plotting of speed-time-distance curves for the line, with estimated current consumption at all points, and the resulting power station, substation and distributing system capacity needed; preliminary designs for a double-track tunnel under Boston Harbor and a terminal at Post Office Square; studies of population density throughout the territory and the number of buildings per acre in disputed districts, and the preliminary design of permanent way and bridges, with cost estimates of grade-crossing elimination. There have also been considered the cost of electrifying the Boston & Maine, and Boston, Revere Beach & Lynn competing steam lines, the possible schedules on the latter and limitations of the former; the present running times and stops of the Boston & Northern Street Ry. Co.'s Boston-Salem line; the preparation of a plan of all special work and yards on the Eastern Division of the Boston & Maine, by personal observation, with different vertical and horizontal scales, to show the difficulties from the petitioner's standpoint of electrifying this division; the plotting of a detailed train sheet and computation of the operating expenses of the new road from the basic figures of train crew and transportation wages in general; cost of power at the motors; cost of repairs, inspection and administration. Finally, a study has been made of the differences between a high-speed interurban electric road operated on a private right-of-way and an electrified steam road; with a study of the New York Central and New Haven electrifications so far as they are suggestive to the foregoing comparison.

The wisdom of such thorough preparation needs no comment, for any project involving an estimated cost of $11,000,000 deserves profound study. It is significant that the companies already in the field have not attempted to any degree to controvert by rebuttal expert testimony the main contentions of the Boston & Eastern engineers as to the possibilities of such a road in the territory at stake. No engineer of wide experience in heavy, high-speed electric traction on roads operated upon an exclusive right-of-way has been brought forward by the existing steam and electric railway companies in the territory to controvert in detail the estimates and calculations of the promoters. The case illustrates that the preparation of an engineering enterprise under modern conditions may carry with it all the elements of an important action at law; it shows how futile is any training of engineers whose curriculum is limited by purely technical problems, and it demonstrates the economy of preliminary expenditures commensurate with the total final estimated cost of a large engineering project.

Notes and Comments.

The Destruction of a Dam across Fall River a few miles below Hot Springs, S. Dak., occurred recently, and is particularly instructive because the failure resulted from the sliding of parts of the structure on the bedrock, without overturning. This dam was described in The Engineering Record of May 9 of this year. The bedrock is hard sandstone, and anchor bolts 1½ in. square and 3 ft. long, projecting equally into the bedrock and the concrete of the dam, were spaced about 6 ft. apart over the foundation. The flood that caused the damage was not unprecedented, but its force was largely augmented by the failure of an old timber dam a quarter of a mile upstream, which the concrete dam was intended to replace. This old dam had not been removed, however, by the company owning both structures, and its failure is considered to have undoubtedly contributed largely to the cause of the accident to the concrete dam, which might have stood otherwise. The structure broke into four pieces. One end section, with the buttress and flume leading to the power house, is intact, and so is the section at the other end of the structure. The central part of the dam broke into two sections, and these slid downstream several feet. The local impression is that some seams in the sandstone of the river bed were opened by the high head of the flood enough to allow an upward pressure to come upon the base of the concrete, and thus enable the impounded water to slide the structure downstream. Accidents of this nature are unusual, but they demonstrate the importance of the absolute tightness of foundations for dams and of bonding masonry and bedrock so thoroughly as to form one mass. In this connection it might also be added that in low dams of this section, which was illustrated in the article referred to, the downstream toe must be regarded as a danger point, for scour is likely to occur below it and work backward under the concrete, if the bedrock is liable to erosion. In the most famous accident of this sort, that at the Austin dam, it was found that such scour had occurred to a considerable extent.

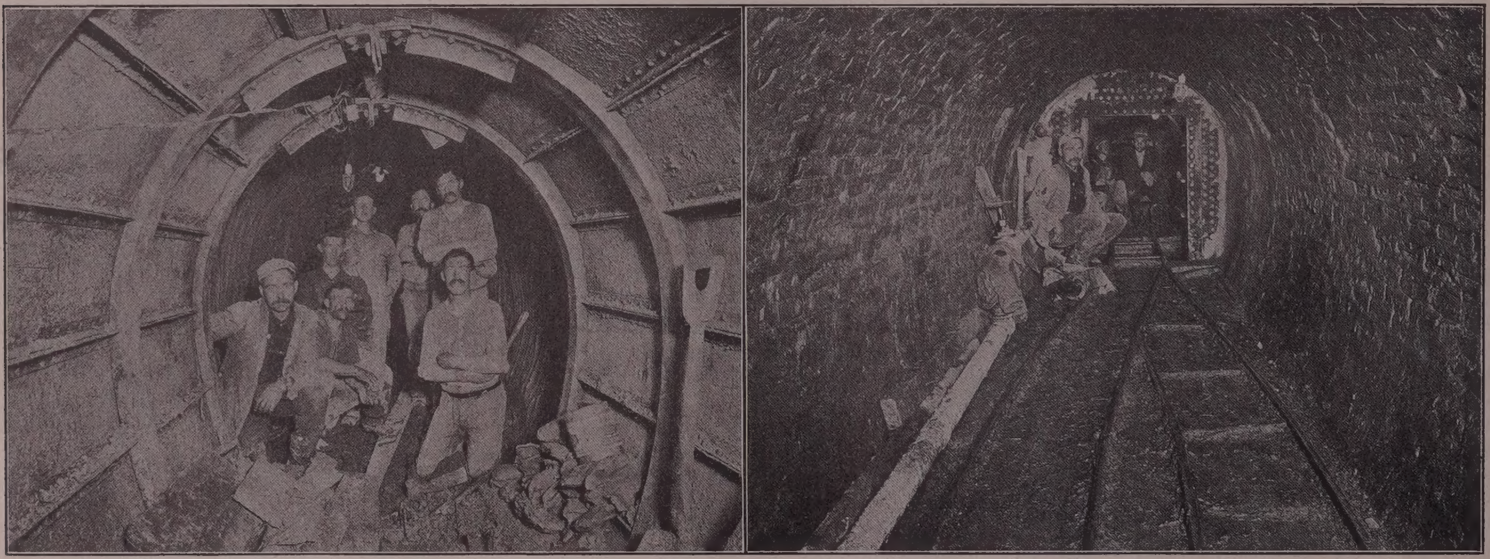
The Pollution of New York Harbor is at last to be investigated in a thorough manner, after several years of preliminary discussion that finally has led to a reorganization of the Metropolitan Sewerage Commission and the voting of $75,000 for conducting its investigations. There was such a divergence of opinion concerning what should be done among the members of the old commission that Mayor McClellan decided, after careful consideration of the subject, that a change in the personnel was necessary, so as to have a commission able to act harmoniously. The subject is particularly important because of the strong feeling in influential quarters that the harbor and bay of New York are already receiving more sewage than it is safe to permit to enter it, while the proposed discharge into the bay of the sewage of the large cities and towns of the Passaic valley is arousing strong opposition. The time has manifestly come when the actual facts regarding the present pollution of these waters and of the effects of increased volumes of sewage discharged into them must be determined accurately, for if the pollution of these waters ever comes before the courts definite facts on the subject must be available. This was well shown by the litigation over the alleged pollution of the water supply of St. Louis by the Chicago drainage canal, and consequently the authorities of the City of New York have shown commendable foresight in determining to begin the investigation now. The members of the commission are all men well qualified for directing the work, and as soon as an engineer to take charge of the details has been selected the investigation will be commenced. The general problem of discharging crude sewage into the waters about New York will shortly become one of the most serious sanitary subjects in the country, and consequently the work of the commission will be watched with unusual interest.

The Floods in the Mississippi Valley, which have caused a great direct and indirect loss, again draw attention to the crude conditions that exist in many places in that great territory. The annual repetition of these reports of ruin and waste occurs so regularly that probably a good many people have come to regard damage by flood as an inevitable condition of existence in some sections of the Valley. Probably several generations will have to bend their energies to the problem, and damage must occasionally happen when extraordinary floods occur, yet it is reasonable to anticipate that a union of engineering ability already available and public assistance on a large scale, resulting from the deep interest now shown in the conservation of our water resources, will eventually reduce this annual loss very materially. The problem is a great one, not to be solved by adopting a few specious technical theories, but only by the thorough study of specialists, who will bring together all the essential facts concerning the runoff of the several regions where the floods originate, the hydraulics of the rivers, the improvement of these streams for navigation and power, and the various means of checking heavy runoffs at different places and of protecting the lowlands. It must be added, moreover, that the recent damage done by floods in some places shows how little attention communities will occasionally pay to the warning of engineers regarding the conditions producing floods. A few years ago, for instance, after a particularly disastrous flood at Kansas City, a detailed study of the river conditions there was made by a board of engineer officers of the army, which pointed out that a repetition of the trouble must be anticipated so long as the natural channel of the river remained obstructed by bridge approaches and similar restraining works. This warning has been unheeded, and there has been an annual record of damage by floods ever since, which would seem to indicate that the city is rather proud than otherwise of its distinction as a flood sufferer.

The Concrete Association of America has made a deserved success of its admirable exhibition in the Brunswick Building, near Madison Square, New York, and has demonstrated the value of such displays in showing the advantages of concrete for various types of construction. A general account of the exhibits was published in this journal last December, but since that time a marked improvement has been effected in many of the exhibits, and others have been introduced. Moreover, some of the exhibitors who took small space at first have learned that the educational work accomplished in these rooms is of decided advantage, and consequently are increasing their space. The idea that a permanent exhibit of concrete work would be profitable was not received with much favor at first by those engaged in the industry, but six months' experience has shown that such an exhibit in New York is one of the most useful methods of presenting to architects and builders the great range of useful purposes to which the material can be put. Here are exhibits by which the architect can show to a client, without leaving the four walls of a large hall, the leading systems of reinforced-concrete structural work, a great variety of fireproof floors and partitions, samples of different methods of treating concrete surfaces for exterior and interior walls, a great variety of artistic moldings, decorative objects in colors, and in a beautiful uniform white as attractive as Italian marble, and hundreds of photographs of completed buildings of every type. There are also numerous exhibits of supplies useful in connection with concrete buildings. Special attention is paid by the Association's representatives to explaining those methods of using concrete which will produce the results desired by visitors in the most economical manner. A great deal of money has been wasted in constructing concrete houses by overlooking features of design which save large sums, relatively speaking, when they are adopted. Too many attempts have been made to follow details of design that have proved useful with wood and brick in connection with concrete, and consequently the cost of comparatively small concrete houses has often been too great. The Association aims to prevent this by showing samples of construction and by affording information that will indicate the most satisfactory methods of design and construction. Another feature of the Association's exhibits that is a source of surprise to most visitors is the collection of panels and independent objects illustrating what can be done with concrete to secure artistic effects; no engineer or architect should fail to visit the hall in order to examine carefully these parts of the exhibition, for they deserve consideration from all who may be called upon to design concrete works where decorative treatment is desirable.

THE INTAKE TUNNEL OF THE MUNICIPAL WATER-WORKS AT GARY, INDIANA.

The municipal water supply for the industrial town which is being built in connection with the immense new works of the Indiana Steel Co. at Gary, Ind., will be obtained from Lake Michigan through a tunnel extending inland from an off-shore submerged crib. This tunnel has a horseshoe-shape cross-section equivalent to a 6-ft. circle, and will have a total length of approximately 15,000 ft. The lake end of the tunnel at the crib is 7,620 ft. from a working shaft on the shore of the lake. The tunnel will extend 7,380 ft. inland from this shore shaft to a shaft under a pumping station in the city, from which station the distribution mains of the water-works system will be supplied. A general description of this system was included in a comprehensive article on the municipal works of Gary in The Engineering Record for July 20, 1907.



Steel Ribs and Lagging in North Drift and Air Lock in South Drift of Shore Shaft.

Bed-rock in the vicinity of Gary lies at a depth of 100 to 120 ft. below Chicago datum, the latter being approximately at the mean level of the lake. Numerous borings made along the line of the tunnel indicate that the rock is overlaid to a depth of 6 to 20 ft. with a stratum of hardpan. Over the latter is a deep stratum of blue clay, varying in stiffness from the hardpan at the bottom to material which flows readily under pressure, and in some cases is not self-sustaining. The thick stratum of clay is covered with a deep bed of sand that is continuous from the top of the clay to the bottom of the lake, and to the surface of the ground. The lake bottom drops 40 ft. in the first 3,000 ft. from the shore, and then continues practically level at that depth to beyond the site of the submerged intake crib. Inland from the lake the surface of the ground was originally a series of sand dunes, generally parallel with the shore line, and with small valleys between these dunes. The latter rose to an extreme height of about 30 ft. above the level of the lake, while some of the valleys were within a few feet of that level.

The tunnel is being driven through the thick stratum of clay, with its invert at a depth of 88 ft. below datum, and 20 to 30 ft. above bedrock. The preliminary borings and the subsequent work in the tunnel show that the character of the clay through which the tunnel headings are advancing is quite irregular, as the stiffness of the material has been found to vary in a short distance from hardpan that requires light blasting to remove, to plastic clay resembling soft putty. As far as the work has progressed no pockets of sand have been encountered in the tunnel headings, nor has any appreciable quantity of water occurred. The ground-water level in the deep bed of sand overlying the clay is, however, approximately the same as that of the lake at the latter, and rises on a flat slope inland until it is at the surface in many places adjacent to the pumping station.

The shaft at the pumping station has an internal diameter of 18 ft. to a depth of 38 ft. below a floor around its top, the latter being 8 ft. above Chicago datum and 3.5 ft. above the maximum level of the lake. To this depth the shaft has a lining of brick masonry, 21 in. thick, laid in Portland cement mortar. A ¼-in. steel-plate core is built in the masonry to a depth of 14 ft. from the top to shut off any flow of ground water. From the bottom of the brick masonry down to the tunnel the shaft is lined with concrete and has an internal diameter of 10 ft. The water will be drawn from the tunnel and elevated to a 300,000-gal. steel stand-tower adjacent to the station by motor-driven centrifugal pumps placed on the pumping station floor, 5 ft. above the level of the lake.

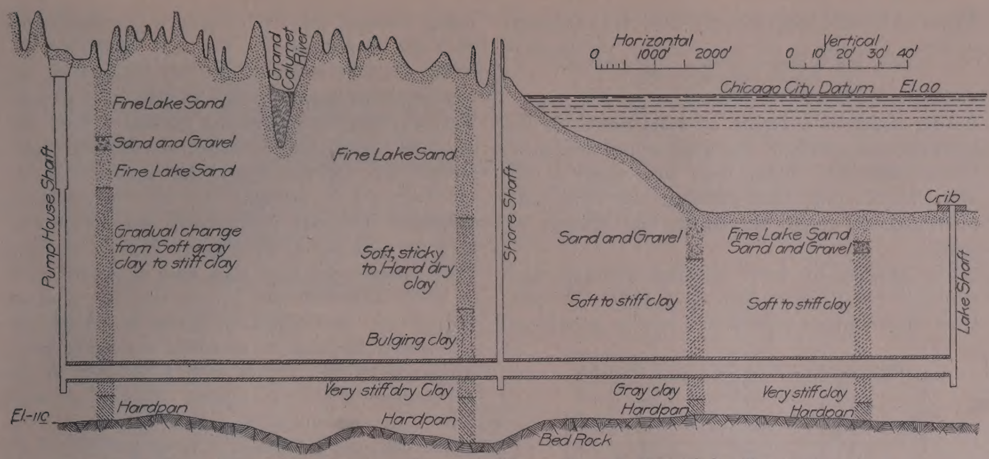
The submerged crib at the lake end of the tunnel is a timber structure, having a total height of 10.5 ft., and is octagonal in plan, with extreme dimensions of 60 ft. It will rest on a foundation of seventy-six round timber piles driven into the bed of the lake and cut off flush with the surface of the sand after the latter has been leveled. The crib will be towed to a place from South Chicago, about twelve miles from the site, where it is under construction, and lowered by means of rock ballast in pockets provided in it for that purpose. A shaft will then be driven through a circular well at the center of the crib down to the level of the tunnel to enable a heading to be started toward the shore.

The shaft at the shore has been built chiefly for construction purposes, but will contain a sluice gate by means of which the flow through the section of the tunnel between it and the pump shaft may be controlled. This working shaft is lined to a depth of 38 ft. below datum with cast-iron cylindrical sections, having a diameter of 10 ft.; below that depth the lining is of concrete, and also has an internal diameter of 10 ft.

The pump shaft and the shore shaft had been sunk, and tunnel headings were being advanced from both of these shafts when these notes were prepared. A single heading was being driven toward the lake from the pump shaft, while work was in progress in two headings started from the base of the shore shaft, one of these going toward the station and the other out under the lake. On May 16 about 2,000 ft. of tunnel had been driven from the pump shaft, 800 ft. from one of the headings at the shore shaft, and 1,500 ft. from the other heading at that shaft. The methods of handling the work had been developed, however, and the organization of the forces had been perfected according to the plans which it is proposed to follow in the remainder of the work.

The sinking of the pump shaft was accomplished without much difficulty, notwithstanding the hazardous conditions existing at its site. A circular steel shoe, with a cutting edge having the same diameter as the outside of the brick lining of the shaft, was first placed in a slight excavation at the surface, and then the masonry lining was built up on this shoe until it reached a height of 10 to 20 ft. A sufficient load of sand in bags was placed on the top of this masonry to force the cutting edge down as the sand was excavated from the enclosed space. When the top of the masonry about reached the ground level, the load was removed and the brickwork again carried up. Working in this manner the cutting edge was carried through the sand and 13 ft. into the underlying clay. The ground-water level being close to the surface, the excavation was made with a clam shell bucket, without lowering the water level by pumping in order to decrease the friction in sinking the shafts. The penetration obtained in the clay with the cutting edge was sufficient, however, to seal the shaft against an inflow of the ground-water under the bottom of that edge. The excavation for the remainder of the shaft, which portion was 10 ft. in diameter, could therefore be sheeted with 4- and 6-ft. pieces of 3-in. wooden lagging, placed vertically and held in place by three horizontal-iron rings to each course of lagging. When the excavation reached the required depth, the concrete lining was started at the bottom and carried up to the brick, the lagging being left in place, although the iron rings holding it were removed as the concrete reached them.

A section of tunnel about 225 ft. long was first driven from the bottom of the pump shaft and lined without the use of compressed air in the heading. An air lock was then installed at this point, and since then all of the operations in the heading have been handled under pressure. The work in the heading is prosecuted by three 8-hr. shifts of laborers. The mining is done by two successive shifts, which advance the heading 24 to 27 ft., under normal conditions. The concrete lining for the bore of the tunnel opened by these two shifts is placed by the third shift, so 24 to 27



Profile of Tunnel Showing General Character of Materials Encountered.

ft. of lined tunnel are completed every 24 hr. in this heading.

The materials encountered in the heading have varied greatly, and at times clay of greatly different character occurs in remarkably short distances. Most of the clay removed, however, is of such nature that it can be cut with a drawknife, although sharp spades and adzes are occasionally used. Four miners and two muckers are worked to each shift engaged in tunneling. The spoil is loaded into ½-yd. cars, with bodies having loose sides and ends, and is pushed through the air lock to a hoisting cage in the pump shaft, in which cage the cars are raised to a truck on a trestle leading out on a spoil bank adjacent to the mouth of the shaft. The excavation is not timbered as it progresses, since an air pressure of 12 to 16 lb. is sufficient to maintain the shape of the bore until the concrete lining can be placed.

The concrete is made, in the proportions of 1 part Portland cement, 2 parts sand and 4 parts broken stone, in a Smith mixer, set up at the mouth of the shaft. The mixer discharges into the ½-yd. spoil cars in which the concrete is carried to the point where work on the lining is in progress. The forms for the concrete lining consist of pieces of 3-in. wooden lagging, 3 ft. long, which are held in place by T-iron ribs bent to the horseshoe-shape of the tunnel cross-section. These ribs are spaced 3 ft. apart on centers. They are made in three pieces, with a joint at the center of the arch, and one at each side of the invert, so they can readily be removed after the concrete is considered to have set sufficiently, which is never sooner than 72 hr. after the lining is placed. During the time concrete work is in progress the air pressure is generally maintained 2 to 4 lb. above that required while the mining is being done.

Air under pressure is supplied to the heading by a 12 x 18 x 12-in. Laidlaw-Dunn-Gordon compressor in a building close to the mouth of the shaft. This building also contains a double-drum Mundy hoisting engine, used in operating the cage in the shaft, and two boilers, one of 110-h.p., and the other 40-h.p. capacity, which supply steam to the compressor, the hoist, the concrete mixer and a pump in a sump at the bottom of the shaft.

The conditions encountered in sinking the shaft at the shore of the lake were such that two unsuccessful attempts were made before a third shaft was finally completed. The ground-water level is within 10 ft. of the surface in the 40 to 50 ft. of fine lake sand that overlies the clay at the site of the shaft, and the latter is close to the edge of the water in the lake. The underlying clay in this vicinity is quite soft, so that it swells and flows when not confined. The cast-iron cylinders that form the lining for the completed shore shaft to a depth of 66 ft. have a shell 2 in. thick, and are each 5 ft. high, weighing about 10 tons apiece. Both ends of each section have flanges inside the shell, with bolt holes in these flanges to enable the adjacent sections to be attached together. The joints between these flanges were made tight by placing strands of oakum and a piece of ¾-in. lead pipe outside the ring of bolts in each of them, and then drawing the bolts up tight enough to press the sides of the lead pipe together and to cause the lead to fill all the irregularities in the faces of the joint.

The first one of the cast-iron cylinders set was built with a sharp bottom edge to assist in forcing the lining through the clay under the sand. The successive sections were attached as the lining was carried down through the sand by removing the latter from the enclosed space with a clamshell bucket, as was done in making the excavation at the pump shaft. The top of the lining was kept high enough above the surface to permit the loads necessary to force the shell down to be applied to it. When the cutting edge reached the clay, the resistance to the progress, on account of the friction on the sides of the shaft, became very great, in fact, the lining was only lowered after the last section was added, by placing 300 tons of ballast on a platform on top of it. This load consisted of pig-iron, sand in sacks, one of the cast-iron cylinders, a standard-gauge flat car and a boiler filled with water. Water jets were also safely applied around the shell above the clay line to relieve the friction of the sand against the shaft, this, of course, being equivalent to that much additional load. The cast-iron lining was finally carried 14 ft. into the soft clay in this manner, which was sufficiently below the sand to cut off the ground-water from the excavation. As rapidly as possible the shaft was then dug through the clay to the level of the tunnel, with a lining of wooden lagging and iron rings placed as it progressed, to hold the clay temporarily. This part of the shaft was next immediately lined with 1 ft. of concrete before the soft clay could collapse the wooden lagging.

Before an air lock was built in either of the two headings driven from the bottom of this shaft, these headings were each advanced 100 ft., from the shaft by lining the tunnel excavation with brick masonry almost as fast as it was opened. Beyond the two locks, the work in the two tunnel headings is handled according to practically the same methods employed in the heading driven from the pump shaft. The spoil is also removed and the concrete delivered in the same manner, two successive 8-hr. shifts mining and the third shift placing the lining for the section opened. The concrete forms in one of these headings, however, consists of Jackson steel ribs and lagging, which have been used quite successfully. The materials thus far encountered in both of these shore shaft headings have generally been much softer than that passed through by the pump-shaft heading, thus requiring 2 to 6 lb. more air pressure in these two headings. At the same time, the progress in the latter has been somewhat greater, since 30 to 33 ft. of tunnel is finished in each heading every 24 hr.

The construction plant at the mouth of this shaft includes a Smith mixer, for the concrete work; a Lidgerwood compressor and a smaller Ingersoll-Rand compressor, to maintain the pressure in the headings, and three boilers, one of

Foot of the Elevator in the Land Shaft and Air Lock in Heading from that Shaft.

125-h.p. and two of 70-h.p. capacity, which furnish steam to this various equipment, and to a pump at the bottom of the shaft.

The submerged crib will soon be launched, towed to the site and sunk in place, after which it is expected to drive the tunnel from a fourth heading. Meanwhile, the high rate of progress that has been obtained in the three headings now under way indicates that the tunnel will nearly be completed in contract time, Dec. 11, 1908, notwithstanding the unexpected delays introduced by the difficulty in obtaining a shore shaft.

The municipal works of Gary are being constructed under the direction of Mr. G. G. Thorp, vice-president of the Indiana Steel Co. Mr. A. B. Neumann is chief engineer, and Mr. W. P. Gleason is general superintendent for that company. Messrs. John W. Alvord and Chas. B. Burdick, consulting engineers of Chicago, designed the water-works and sewerage systems, and are supervising the construction of those systems. Mr. W. F. Sargent is engineer in immediate charge of the construction of these systems. The intake tunnel and submerged crib are being built by the Great Lakes Dredge & Dock Co., of Chicago, who are contractors also for the construction of Gary harbor, the progress and methods of which work were described in The Engineering Record for Aug. 17, 1907.

A Pony Truss Skew Bridge.

The Ellsworth Ave. Bridge, recently erected over the tracks of the Pennsylvania R. R., in Pittsburg, Pa., is an instance where excessive skew caused the use of a pony truss. It was first intended to use top bracing, but difficulties in the top and in the portal bracing, the horizontal strut of which would have been over 90 ft. long, resulted in the abandonment of such bracing. The skew is 82 ft., and the angle 22 deg. 28 min., resulting in placing the extreme shoes of the opposite trusses 267 ft. 7 in. apart, measured parallel to the axis of the bridge. The trusses are 185 ft. 7 in. long, center to center of shoes, 34 ft. apart, 20 ft. high at the center of the span and 10 ft. at the ends, with top chords curved. The highway traffic is heavy, and the bridge carries two street-railway tracks. The floorbeams are only 2 ft. 3 in. deep, supporting a buckle-plate floor. Footwalks are cantilevered out on both sides.

The new bridge replaces an older structure, with its trusses in the same position. Since the old bridge was strong enough to carry the new trusses, and it was undesirable to drive a falsework of pile bents between the railroad tracks below, the old trusses were used to support the new ones during erection. When the latter had been completed, they were slid to position and the old trusses dismantled.

Tests of Bond in Reinforced Concrete Beams.

A paper presented at the convention of the American Society for Testing Materials, by Mr. Morton O. Whitey, Madison, Wis.

This paper is a report of tests made on the bond between concrete and plain round mild steel reinforcing bars. These tests were made in the Materials Testing Laboratory at the University of Wisconsin during the spring and summer of 1907.

The subject of bond between concrete and steel has led to many extensive series of tests. Most of these tests have been made by embedding a rod in the center of a cylindrical block of concrete and pulling it out in such a way that the concrete around the rod was compressed. In general, the results of such tests have neither given conclusive nor reliable data upon which working values for structural purposes can be based.

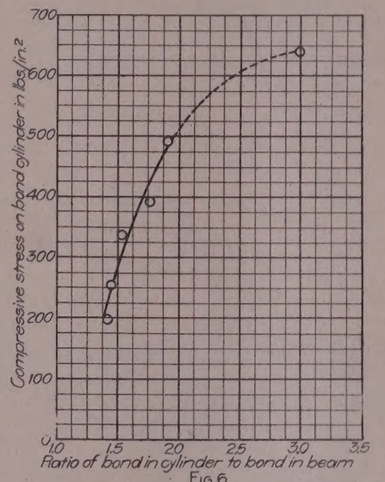


Fig.6

Fig. 6.—From Values in Table 7.

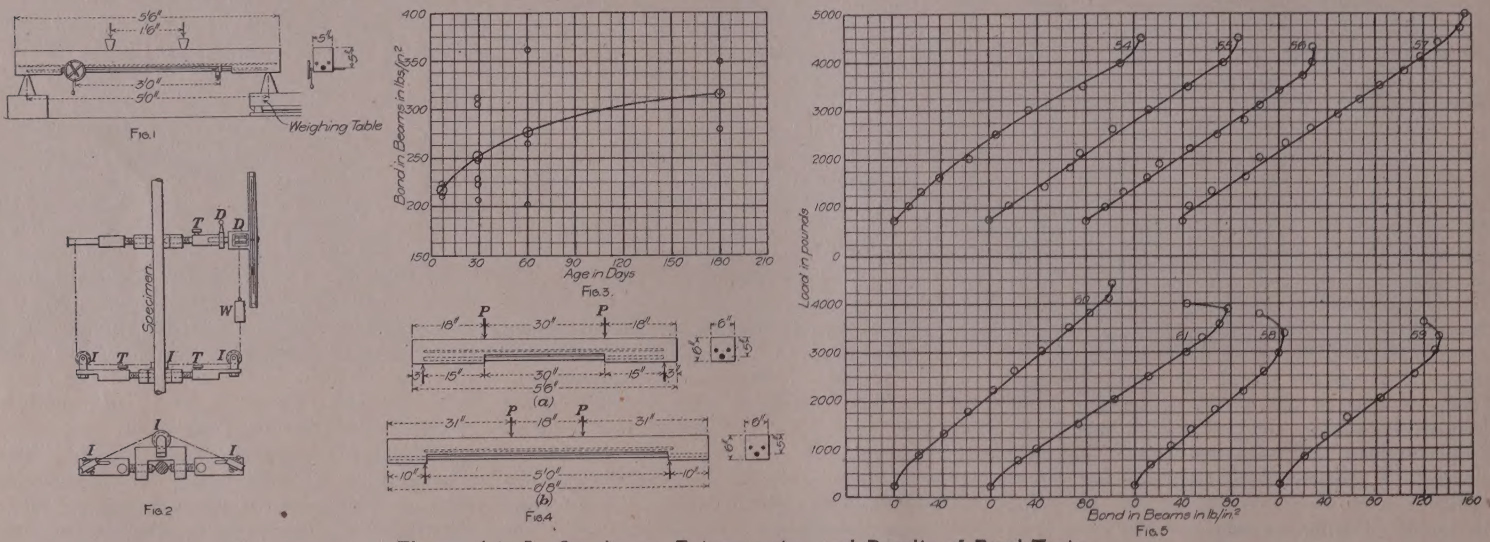


Fig.1 Fig.2 Fig.3. Fig.4 Fig.5

Figures 1 to 5.—Specimens, Extensometer, and Results of Bond Tests.

In order to approximate more closely the actual conditions in reinforced concrete beams, a different form of bond specimen was used in the tests herein described and the results compared with those obtained from the ordinary cylinder test. The larger portion of the specimens were made and tested as thesis work by Messrs. A. B. Carey, S. L. Clark and P. B. Johnson, of the class of 1907, to whom due acknowledgment is hereby made.

The sand used in these tests was a rather fine local bank sand. It weighed 103 lb. per cubic foot, measured loose, contained 37.5 per cent. voids and 3 per cent. of fine dirt.

The limestone used was obtained from Kankakee, Ill. It would pass a 1-in. mesh, weighed 83 lb. per cubic foot and contained 48.5 per cent. voids. A local bank gravel was used in beams 19 and 20, particles less than 0.2 in. in diameter being screened out. It weighed 105 lb. per cubic foot and contained 35½ per cent. voids.

The Atlas cement from Missouri took an initial set in 45 min., final set in 5 hr. and 40 min., and satisfactorily passed the pat tests. At 7 and 28 days the average strength of four neat briquettes was 529 and 659 lb. per square inch, respectively. The strength of the 1:3 mortar briquettes increased from 204 lb. per square inch at 7 days to 309 lb. per square inch at 28 days. Plain round mild steel rods with an elastic limit of about 44,000 lb. per square inch, an ultimate strength of approximately 64,000 lb. per square inch and a modulus of elasticity averaging about 30,000,000 lb. per square inch were used as reinforcement in the test pieces. These rods were free from rust and loose scales.

The concrete for beams 1 to 49 was mixed by hand on the cement floor in the laboratory. The rest of the beams were mixed by machine. Materials were proportioned by volume. Quantities required were weighed on a platform scale. The weight of the cement was assumed to be 100 lb. per cubic foot.

Three consistencies were used in beams 23 to 31, wet, medium and dry. The wet concrete was sloppy, could not be piled up on the mixing floor, and when placed in the molds showed an excess of water on top. The medium concrete when piled on the floor would flatten out, could be readily worked around the reinforcement without much tamping and was easily surfaced with a trowel. The dry concrete was very mealy, would stand piled up on the floor, required hard tamping and could not be readily surfaced with trowel. The medium consistency was used in all other beams.

The form of specimen adopted for determining the bond in beams is shown in Fig. 1. These beams were in general 5 ft. 6 in. long, 5 in. wide and 6 in. deep. They were so made that the central portion of the lower rod, whose bond was to be obtained, was exposed. The two ⅜-in. round rods seen in the figure were placed above the exposed bar to distribute the tension cracks and prevent failure in the concrete before the lower rod slipped. Wooden forms were used in making beams 1 to 49, steel forms in the remainder.

In beams, the exposure of the lower rod was secured by covering it with damp sand before the concrete was placed. A notched piece of wood placed transversely with respect to the axis of the beam served to space the rod and provide a definite limit for the predetermined length of embedment. The embedment used for all beams with lower rod ⅝ in. in diameter or larger was approximately 10 in., except in beams 62, 63 and 64, where it was approximately 15 in. The embedment of the lower ⅜-in. rod in beams

38, 39 and 40 was nearly 7½ in., and that of the ½-in. rod in beams 41, 42 and 43 about 8½ in. After each test, the actual length of embedment of the end at which slipping occurred was carefully measured. This quantity was then used in obtaining the results given in the tables.

Auxiliary compression specimens consisting of 4-in. cubes or cylinders 6 in. in diameter and 18 in. long were made with nearly every batch of concrete as indicated in Table 1. The bond cylinders were also cast from the same batch of concrete as the corresponding beams. In all cases they were made with the rod vertical and an embedment of practically 12 in.

TABLE 1.—DATA ON SPECIMENS MADE.

Beam No.	Mix.	Consistency.	Size of Rod.	Age.	Variable Studied.	Kind of Auxiliary Specimens.
1-2-3.	1:2:4	medium	⅝ in.	7 da.	time	compression
4-5-6.	1:2:4	medium	⅝ in.	28 da.	time	compression
7-8-9.	1:2:4	medium	⅝ in.	60 da.	time	compression
10-11-12.	1:2:4	medium	⅝ in.	6 mos.	time	compression
7-8-9.	1:2:4	medium	⅝ in.	60 da.	mix	compression
17-18.	1:3:6	medium	⅝ in.	60 da.	mix	compression
* 19-20.	1:2:4	medium	⅝ in.	60 da.	mix	compression
21-22.	1:2½	medium	⅝ in.	60 da.	mix	compression
23-24-25.	1:2:4	wet	⅝ in.	60 da.	consistency	
26-27-28.	1:2:4	medium	⅝ in.	60 da.	consistency	
29-30-31.	1:2:4	dry	⅝ in.	60 da.	consistency	
36-37.	1:2:4	medium	⅝ in.	60 da.	method of test	bond
38-39-40.	1:2:4	medium	⅜ in.	60 da.	size of rods	bond
41-42-43.	1:2:4	medium	½ in.	60 da.	size of rods	bond
44-45-46.	1:2:4	medium	¾ in.	60 da.	size of rods	bond
47-48-49.	1:2:4	medium	1 in.	60 da.	size of rods	bond
54-55-56-57.	1:2:4	medium	⅝ in.	28 da.	time and loading	compress'n
58-59-60-61.	1:2:4	medium	⅝ in.	28 da.	loading	compress'n
62-63-64.	1:2:4	medium	⅝ in.	29 da.	loading	compress'n and bond.

*In 19 and 20 gravel was used in place of limestone.

Beams 1 to 49 were kept in the molds for at least three days after mixing. They were then allowed to cure without being dampened until tested. Beams 50 to 61 were kept in the forms two days and sprinkled every day for a week after mixing. Beams 62, 63 and 64 were kept in the forms three days and sprinkled twice a day until tested. This continued wetting may have weakened their bond strength. Table 1 gives the general data on the 56 beams which were made.

All beams were tested over a 5-ft. span. All beams except 62, 63 and 64 were loaded as shown in Fig. 1. Fig. 4(*b*) indicates how beams 62, 63 and 64 were loaded and also shows the different shape adopted for beams 58 to 61. Loads were applied in increments of 200 or 300 lb. and readings of the deformation in the rod taken on the extensometer.

The deformation of the rod was measured by a dial extensometer reading to 0.0001 of an inch, shown in Fig. 2. This extensometer was described in a paper presented before this society last year by Prof. H. F. Moore. It is only necessary to say in regard to it that an average deformation of both sides of the rod was secured from a single reading of the dial. The readings are considered sufficiently accurate for this class of work. As the rod in the beam was horizontal instead of vertical, the weight, *W*, hung down perpendicular to the plane of the paper and not as shown in Fig. 2. Slipping of the rod was immediately indicated by a backward movement of the pointer on the dial. The dropping of small particles of sand and concrete from the surface of the rod where it entered the beam made evident the end which slipped.

The rods embedded in cylinders were pulled out by a universal 50,000-lb. testing machine. The pulling head was raised so that the projecting part of the rod could be run up through it and gripped by the wedges in the upper head. If the top of the cylinder was uneven, blotting paper was inserted between it and the pulling head to equalize the pressure. No allowance was made for the area of the hole in the pulling head, 8 sq. in., in computing the stresses in Table 7.

To calculate the bond in a beam from the deformation of the rod, the following formula was used:

$$B = r d E \div 8 L l$$

where B = bond in pounds per square inch, r = dial reading plus zero correction, d = diameter of rod in inches, E = 30,000,000 lb. per square inch, L = gauge length of extensometer, and l = length of embedment of rod.

TABLE 2.—EFFECT OF TIME ON THE BOND OF 1:2:4 CONCRETE.

Beam No.	Maximum load, lb.	Age.	Stress in steel, lb./in.^{2}	Bond, lb./in.^{2}	Compressive strength of concrete, lb./in.^{2}
1	4,000	7 da.	14,200	222	....
2	3,400	7 da.	13,500	211	....
3	Beam accidentally destroyed.				
Average.	....	7 da.	....	216	1,400
4	6,800	28 da.	23,000	305	....
5	4,400	28 da.	19,200	312	2,030
6	4,600	28 da.	12,900	207	....
54	4,500	28 da.	14,500	228	....
55	4,500	28 da.	15,900	248	....
56	4,300	28 da.	14,100	222	1,690
57	5,000	28 da.	16,300	247	....
Average.	....	28 da.	....	253	1,860
7	4,800	60 da.	18,500	362	....
8	4,200	60 da.	16,900	264	....
9	4,000	60 da.	11,100	201	....
Average.	....	60 da.	....	276	1,790
10	4,200	6 mos.	17,900	280	....
11	5,200	6 mos.	21,300	350	....
12	4,800	6 mos.	16,800	318	....
Average.	....	6 mos.	....	316	1,630

In Table 2 are given the results of beam tests to determine the effect of time upon the bond. Although there was a considerable variation from the mean at some of the ages, the average bond increased with the age of the specimen. This is well shown by the time-bond curve in Fig. 3, drawn through these plotted averages.

TABLE 3.—COMPARISON OF THE BOND OF VARIOUS MIXES. AGE 60 DAYS.

Beam No.	Mix.	Maximum load lb.	Stress in steel, lb./in.^{2}	Bond, lb./in.^{2}	Compressive strength of concrete, lb./in.^{2}
7 ...	1:2:4	4,800	18,500	362	....
8 ...	1:2:4	4,200	16,900	264	....
9 ...	1:2:4	4,000	11,100	201	....
Average	1:2:4	....	....	276	1,790
17 ...	1:3:6	3,200	11,600	181	....
18 ...	1:3:6	4,400	18,100	251	....
Average	1:3:6	....	....	216	830
19 ...	1:2:4 g.	5,200	19,900	282	....
20 ...	1:2:4 g.	4,800	17,200	269	....
Average	1:2:4 g.	....	....	275	2,200
21 ...	1:2½	3,600	16,600	235	....
22 ...	1:2½	3,800	18,200	300	....
Average	1:2½	....	....	267	1,600

Table 3 contains the results of bond tests made on different mixes. The bond for the 1:3:6 mix was decidedly lower than the others. It will also be noted that the compressive strength of this mix was only about half that of the other mixes. There was very little difference in the average bond of 1:2:4 limestone, 1:2:4 gravel and 1:2½ mortar.

TABLE 4.—EFFECT OF DIFFERENT CONSISTENCIES ON THE BOND OF 1:2:4 CONCRETE.

Beam No.	Consistency.	Maximum load, lb.	Stress in steel, lb./in.^{2}	Bond, lb./in.^{2}
23	wet	3,200	11,000	180
24	wet	4,800	22,100	346
25	wet	4,000	16,500	224
Average.	wet	....	....	250
26	medium	3,200	11,800	171
27	medium	3,600	18,500	321
28	medium	3,500	13,600	212
Average.	medium	....	....	235
29	dry	4,800	19,300	287
30	dry	3,600	20,100	314
31	dry	3,600	13,600	224
Average.	dry	....	....	275

Age of specimens, 60 days.

The data for tests upon wet, medium and dry consistencies in Table 4 indicate that the amount of water used in these tests had no decided effect on the bond when the specimens were 60 days old.

TABLE 5.—EFFECT OF SIZE OF ROD ON THE BOND OF 1:2:4 CONCRETE. AGE, 60 DAYS.

Beam No.	Size of rod.	Maximum load, lb.	Stress in steel, lb./in.^{2}	Bond, lb./in.^{2}
38	⅜ in.	3,200	27,600	345
39	⅜ in.	3,400	23,800	298
40	⅜ in.	2,400	16,700	190
Average.	⅜ in.	....	....	278
41	½ in.	4,400	26,400	361
42	½ in.	3,600	21,200	312
43	½ in.	2,800	12,300	186
Average.	½ in.	....	....	286
7	⅝ in.	4,800	18,500	362
8	⅝ in.	4,200	16,900	264
9	⅝ in.	4,000	11,100	201
Average.	⅝ in.	....	....	276
44	¾ in.	5,000	12,400	207
45	¾ in.	4,400	13,900	289
46	¾ in.	6,200	15,700	295
Average.	¾ in.	....	....	264
47	1 in.	3,400	5,850	136
48	1 in.	4,000	7,650	174
49	1 in.	4,000	7,900	180
Average.	1 in.	....	....	163

From Table 5 it appears that the size of the rod up to 1-in. diameter has but little influence on the bond. The specimens with the 1-in. rod showed a marked decrease in bond. More bond tests with this size of bar are necessary before this decrease can be satisfactorily accounted for.

Beams 54 to 61 were made to determine the effect on bond of embedding the rod in stressed and unstressed concrete. Beams 54 to 57 were made and loaded, as shown in Fig. 1. Fig. 4 (*b*) represents beams 58 to 61. The results apparently indicate that the bond of concrete subjected to a tensile stress is considerably greater than that of concrete which is practically unstressed. It is possible that bending increased the resistance against slipping in the first four of these beams. It will be observed also that compressive strength of the concrete for beams 54 to 57 was considerably higher than that tested from beams 58 to 61.

TABLE 6.—MISCELLANEOUS TESTS ON THE BOND OF 1:2:4 CONCRETE.

Beam No.	Maximum load lbs.	Stress in steel lbs./in.^{2}	Bond lbs./in.^{2}	Compressive strength of concrete lbs./in.^{2}
54	4,500	14,500	228	....
55	4,500	15,900	248	....
56	4,300	14,100	222	....
57	5,000	16,300	247	....
Average			236	1,690
58	3,400	8,800	129	....
59	3,300	9,500	141	....
60	4,100	12,600	187	....
61	3,900	13,700	204	....
Average			165	1,070
62	6,850	19,000	197	....
63	6,800	20,300	215	....
64	7,000	22,100	236	....
Average			216	2,315

Beams 62, 63 and 64 were designed to determine the effect of a change from the standard loading, shown in Fig. 1. They were made and loaded as shown in Fig. 4 (*a*). The average bond of these beams did not differ materially from that obtained from beams 54 to 57. The small decrease in bond developed in 62, 63 and 64 might have been due to the fact that they were sprinkled more frequently while curing than the other beams. This might have decreased the shrinkage of the concrete and thereby lessened the bond.

If the bond stress be computed in the ordinary way from the average vertical shear of beams 62, 63 and 64, a value of 214 lb. per square inch will be obtained. In making this computation the arm of the resisting couple was taken as 0.82 times the effective depth. The distance from the top of the beam to the ⅜-in. rods was 3¾ in. This quantity checks very closely the average value obtained from the deformation of the lower rod, 216 lb. per square inch.

Fig. 5 shows load-bond curves for beams 54 to 61. It will be seen that all the curves show a reversal of curvature at the load at which the maximum bond was developed.

TABLE 7.—COMPARISON OF BEAM AND CYLINDER TESTS ON BOND.

Beam No.	Size of rod ins.	Average bond in Beam lbs./in.^{2}	Average bond in cylinders lbs./in.^{2}	Bond in cyl. ÷ Bond in beam.	Compressive stress on top of bond cyl. lbs./in.^{2}
38-39-40	⅜	278	394	1.42	197
41-42-43	½	286	455	1.54	317
62-63-64	⅝	216	311	1.44	252
36-37	⅝	266	467	1.76	395
44-45-46	¾	264	502	1.90	495
47-48-49	1	163	487	2.99	620

A comparison between the bond developed in beams and that obtained in bond cylinders made at the same time is afforded in Table 7. Column 5, headed Bond in Cylinder÷Bond in Beam, shows that the cylinder tests gave bond stresses from 1.42 to 2.99 times as large as those obtained from the beam tests. Fig. 6 is a curve plotted from the values in the last two columns of Table 7. It indicates that the ratio of bond in cylinder to bond in beam increases with the compressive stress applied to the top of the bond cylinder. The upper part of the curve was drawn a dotted line as more tests of 1-in. bars are needed to fix its position definitely.

Conclusions.—1. The bond of 1:2:4 concrete to embedded steel increases with age at least up to six months. About 80 per cent. of its six months bond strength is developed at 28 days.

2. Owing to the variation in results of the one-month tests, and the wide difference between laboratory experiments and practical working conditions, it does not seem as though the maximum bond of 1:2:4 concrete should be assumed greater than 200 or 250 lb. per square inch in designing.

3. The method of making bond tests by pulling a rod from a concrete cylinder, as previously described, gives results which are of neither quantitative nor qualitative value. The results obtained are dependent largely upon compressive stress acting on the head of the cylinder.

4. The beam test for bond on the other hand approaches closely the actual conditions to which the bar and surrounding concrete are most often subjected, and gives values which are at least of qualitative value. Furthermore, values obtained by this method are in accord with the bond theory usually accepted as a basis in design.

These tests have been continued during the past year. The bond of both plain and deformed bars, when subjected to static and repeated loadings, has been investigated, and the results of all of these tests will appear during the year in a University of Wisconsin bulletin.

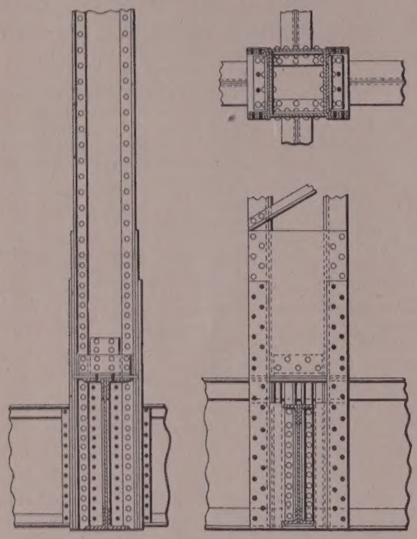
The Section of the Grand Trunk Pacific Ry. between Portage La Prairie, Manitoba, and East Saskatchewan was completed May 14, giving a continuous line 500 miles long.

The California City Point Coal Pier.

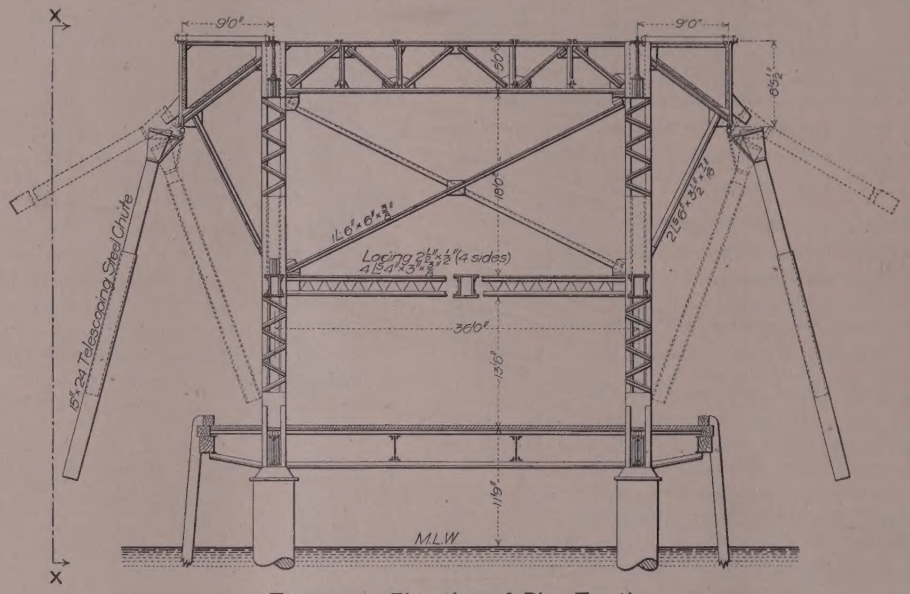
The United States Naval Coal Pier at California City Point is located just opposite the Golden Gate in San Francisco Bay, California, at the foot of a high bluff in a situation exposed to considerable seas which may be raised by the east wind blowing across the full width of the bay. The depot comprises a wooden bunker of about 10,000 tons capacity located on a terrace cut out of the foot of the bluff and served by a system of cable cars delivering the coal to and from a pier 48 ft. wide and 390 ft. long and about 12 ft. above mean low water. The pier is parallel with the shore line and about 150 ft. beyond it, in water nearly 30 ft. deep, with from 4 to 40 ft. of mud overlying a hard bottom of partly decomposed rock.

The pier is supported on iron cylinders braced together to form alternate towers and open panels carrying steel beams and girders and a wooden floor on which there is erected a steel viaduct 36 ft. wide and 36 ft. high, to carry two unloading travelers and four lines of cable cars. Vessels can lie on both sides of the pier and be coaled at the rate of about 300 tons per hour by a gravity system independent of the unloading apparatus, which can be in simultaneous operation with a capacity of 200 tons per hour. Coal can be handled directly from a lighter on one side of this pier to a vessel on the other side, but is ordinarily transferred directly from the lighter to storage in the bunker and then returned by the cable-car system as required for coaling vessels.

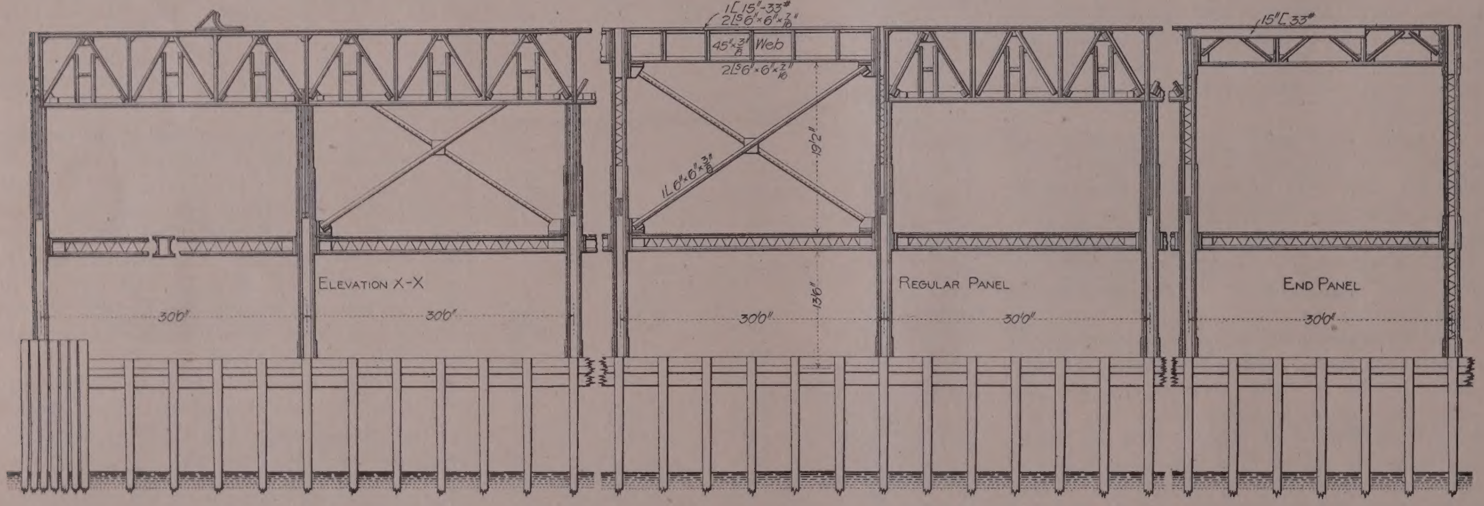
The original plans contemplated the construction of pier cylinders 8 ft. in diameter, made with segmental steel sheet piles driven in position, excavated in the dry and afterwards filled with concrete. These provided no opportunity for lateral and transverse diagonal or horizontal bracing. Owing, also, to their length of about 75 ft. maximum, the weight of the structure and the stress to be apprehended from impact, wind pressure, and waves, this design was decided to be deficient in rigidity and was superseded by the system, shown in the accompanying illustrations, of castiron cylinders braced together with horizontal and diagonal longitudinal and transverse members.



Connection of Viaduct Post to Floorbeam.



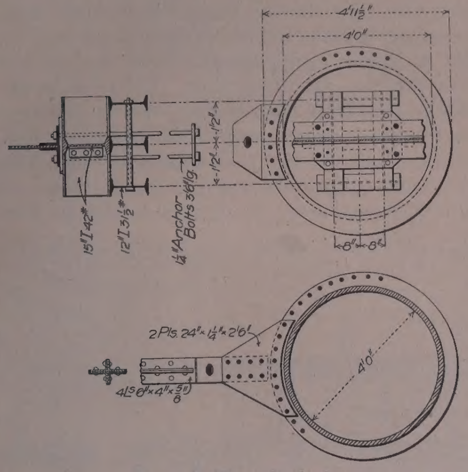
Transverse Elevation of Pier Trestle.



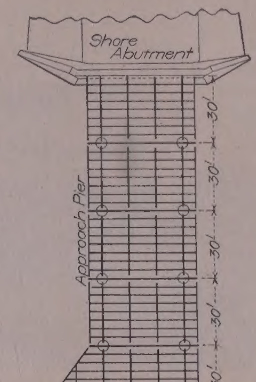
Outside Elevation, Elevation on Center of Cylinders, and End Panel of Coal Pier.

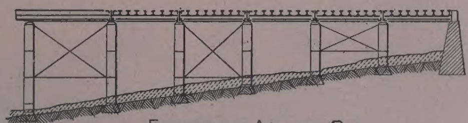
These cylinders are arranged in pairs 36 ft. apart transversely, in bents 30 ft. apart longitudinally, and are braced at the upper ends by the floor beams and stringers. Transverse struts, made with four 6 x 4-in. angles riveted together back to back, are bolted through jaw plates to the flanges of the cylinders 24 ft. below their upper end, and the panels between them and the floor girders are X-braced with sleeve-nut rods having clevis ends. The plate-girder floor beams, nearly 49 ft. long over all, are seated on top of the cylinders and are web-connected to four lines of longitudinal plate-girder stringers and two lines of fascia girders, 30 and 36 in. deep in the center and outer lines, respectively, which carry on their top flanges transverse 8-in. I-beams, about 4 ft. apart, to receive 3-in. longitudinal floor planks. The traveler has a heavy machinery deck about 15 ft. above the rail, and is equipped with hoisting engines and locomotive machinery necessary to operate the booms and buckets and to traverse it from end to end of the pier.

About five hundred and sixty regular 12-ft. lengths of 4-ft. castiron pipe were received for the pier cylinders, besides thirty-six special pieces of variable length calculated to conform to the irregularities of level in the hard bottom, and thirty-six special conical bottom castings. A conical section and one or more long sections, according to the depth of the water, were permanently bolted together and lowered vertically to bearing in the required position in the mud bottom, where they were maintained by guys and by the boom of a stiff-leg derrick mounted on a scow carefully moored and maintained in the position where it was aligned from shore.

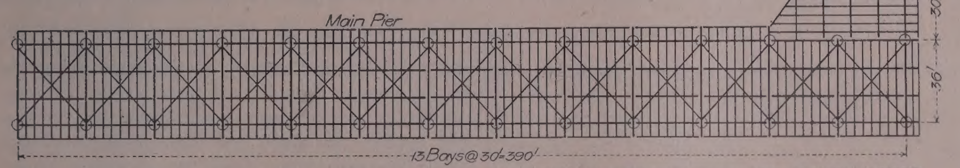


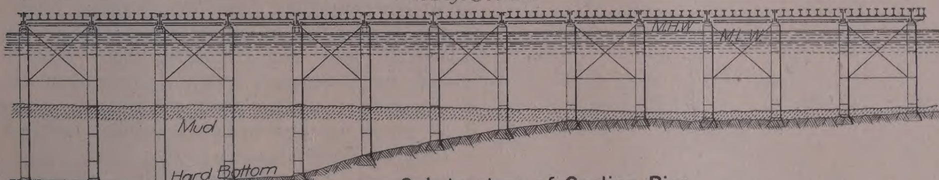
Top of Pier Cylinder.





Elevation of Approach Pier





Substructure of Coaling Pier.

The cylinders usually penetrated by their own weight into the mud sufficiently to insure their stability, while the interior was excavated with a ½-yd. Hayward orange-peel dredge bucket, operated by the Lidgerwood derrick engine. The dredging was continued to the surface of the rock and beyond to satisfactory hard stratum. The weight of the cylinder, augmented by successive sections bolted to the upper end as it descended, was usually sufficient to carry it down as fast as the dredging progressed.

When the cylinders were very long, or the resistance became too great, sinking was facilitated by temporarily loading the cylinder with beams and girders or by the use of a water-jet around the cutting edge. The increased width at the bottom tended to permit lateral diversion of the cylinder, and special pains were necessary to keep it always carefully guyed in order to maintain it in a vertical position until it reached the rock, where it was bedded as accurately as possible. Most of the mud and fine material were then pumped out, and concrete deposited under water with an automatic bottom-dump bucket to fill the conical portion, secure an extended and equalized bearing, and seal the interior against the entrance of any considerable quantity of water. After this concrete had set no difficulty was experienced in pumping the cylinder dry above it, so that men could enter and adjust and arrange the reinforcement steel.

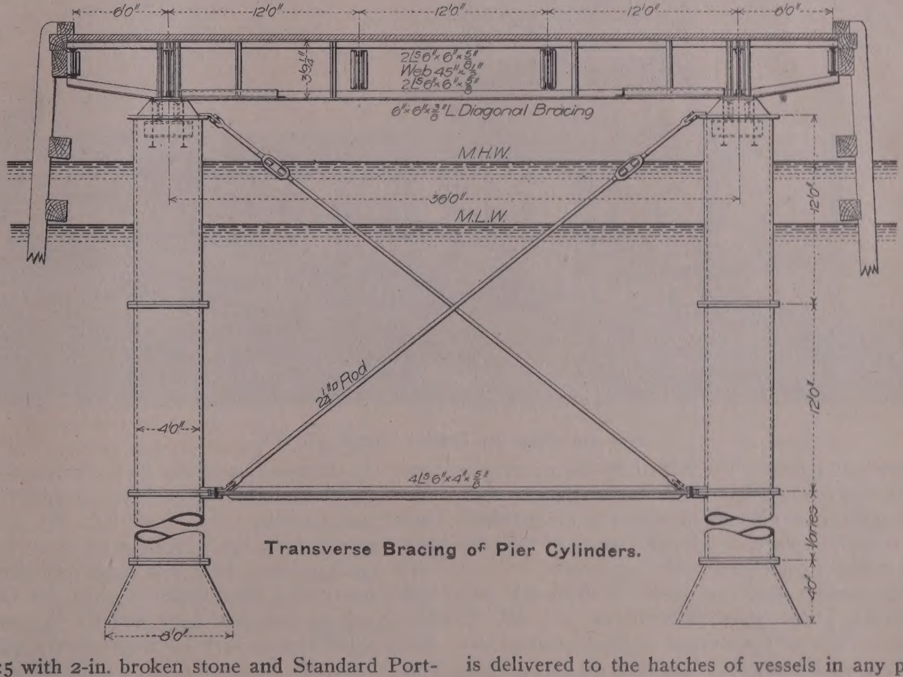
Each cylinder was reinforced with fourteen vertical steel bars 1 3/16 in. in diameter, secured by No. 12 wire, to 12 x ¼-in. horizontal circular bands 6 ft. apart vertically. The rods were made in lengths of from 18 to 24 ft. lapped and breaking joints. After the rods were set concentric with the cylinders, the latter were filled with concrete rammed in 12-in. layers and mixed in Smith concrete machines, from which it was delivered in chutes directly to the cylinders. It was made 1:3:5 with 2-in. broken stone and Standard Portland cement manufactured at San Francisco. The fascia girders and the ends of the floor beams are protected by heavy longitudinal timbers bolted to slightly battered piles, 5 ft. apart on centers, which are secured to heavy inside waling pieces at high and low water level and form an elastic rack to serve as a fender and protect the structure from vessels alongside.

Great economy was effected by making the cylinders from regular commercial sections of standard castiron water pipe 4 ft. in diameter, 1½ in. thick, 12 ft. long, which weigh about 10,000 lb. each and are connected by forty-four 1⅜-in. bolts through each of the exterior flanges. In order to secure the required bearing area at the bottom, the foot of each cylinder is provided with a special conical section 4 ft. long and 8 ft. in diameter at the bottom, thus giving it the same base that was contemplated for the sheet pile cylinders. An I-beam grillage at the upper end of the cylinder is securely anchored to its concrete filling and provides support for the transverse girders. A lateral system is provided just below the deck planks by an X-bracing of single 6 x 6 x ⅜-in. angles connecting the tops of the cylinders in all panels of the main pier.

The steel superstructure carrying the coal tracks, unloading travelers, and loading chutes above the deck of the main pier is made with steel columns 38 ft. long on the centers of the cylinders. The columns have longitudinal and transverse horizontal and diagonal braces forming a series of rectangular towers and open panels continuously connected by the longitudinal trusses and track girders at the tops of the columns. The columns have a rectangular cross-section made with two built channels latticed with small angles; the lower ends of the channel-webs are slotted to clear the webs of the transverse girders, to which they are field-riveted through vertical connection angles, and similar connections are made between their cover plates and the webs of the longitudinal girders.

The upper ends of the columns are connected by transverse lattice girders, 5 ft. deep, and by 45-in. longitudinal plate girders, the latter serving as stringers for the traveler track. A line of special longitudinal lattice girders, 8½ ft. deep, is bracketed out 9 ft. beyond the column centers on each side of the viaduct and forms part of the framework of a continuous loading bunker with a capacity of 1½ tons per lineal foot. This bunker runs from end to end of the pier and is provided with a traveler rail supporting two adjustable telescopic steel chutes 47 ft. long, by which coal is delivered to the hatches of vessels in any position, through gates 10 ft. apart in the side of the bunker. The columns are braced by horizontal longitudinal and transverse struts, 13 ft. above the lower deck, which struts are made with built channels latticed on both flanges, and the panels between them in both faces of the towers are X-braced with single 6 x 6-in. riveted angles.

A very rigid superstructure is thus secured with abundant strength for the two unloading travelers installed on it. Each traveler is a wedge-shape steel tower about 80 ft. in height with a



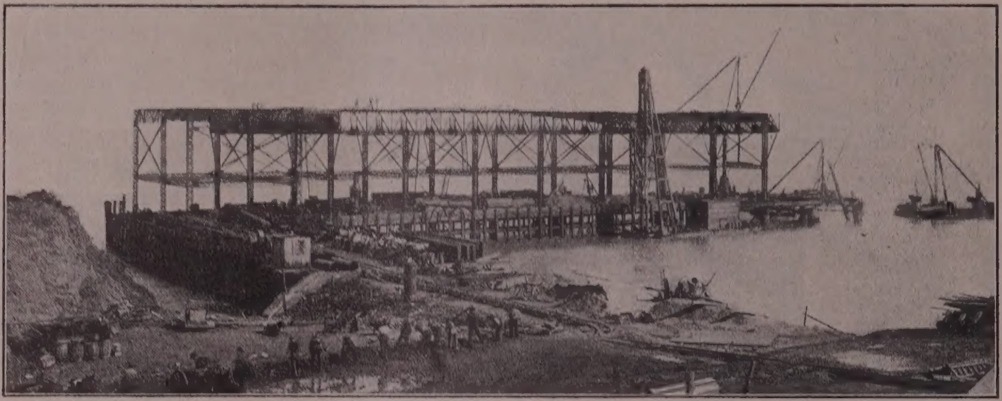
Transverse Bracing of Pier Cylinders.

30 x 36-ft. base, mounted on 16 wheels on the 36-ft. gauge track.

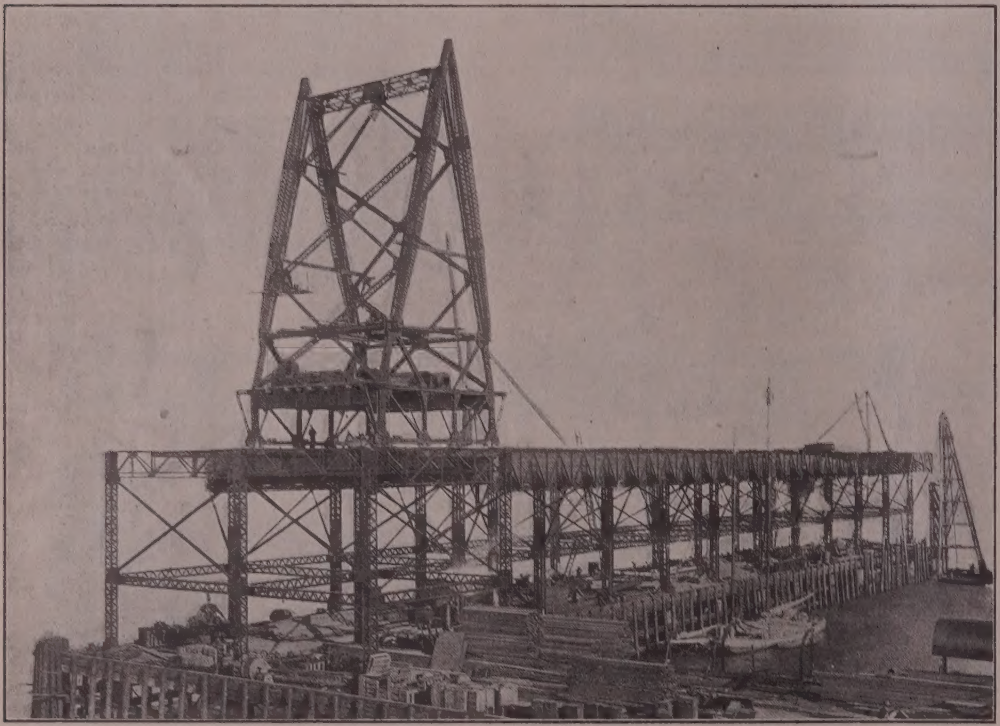
Grillage beams and anchor bolts were embedded in the concrete in the top of the cylinder and the concrete was carried up about 2 ft. beyond the shell to enclose them and to receive a ¾-in. bed plate for the girders. As fast as the cylinders were set in position the lower horizontal struts and diagonal bracings were connected by divers and the spacing timbers by which they were temporarily maintained in position were replaced by the permanent beams and girders of the lower deck, which were delivered from shore by a service track running on the completed structure and were unloaded and erected by a traveling stiff-leg derrick.

After a sufficient length of the lower deck was completed, two panels of the viaduct were erected on it by a stiff-leg derrick seated on the lower platform, where they were braced together to make a stable tower, a derrick was hoisted to its upper platform and then proceeded to erect the remainder, one panel in advance in the manner usually adopted for erecting viaducts on land, with a 10-ton, 40-ft. boom commanding one panel beyond the base of the derrick.

The principal quantities involved in this contract include 800 tons of castiron, 700 tons of structural steel and about 1,000 cu. yd. of concrete. The work was executed by an average force of from 30 to 40 men, and preparations had been made for its commencement when conditions were so much unsettled by the occurence of the disastrous San Francisco earthquake that it became very difficult to secure labor, so construction was postponed for a number of months, being finally completed in March, 1908.



Sinking Cylinders for Coal Pier, February 29, 1908.



Pier and Coaling Tower, March 31, 1908.

The designs were approved and the work was executed under the direction of Mr. W. S. Cowles, chief of the Bureau of Equipment of the U. S. Navy Department. The contract was awarded to the Snare & Triest Co.; Mr. E. A. Sheble, designing engineer; Mr. F. J. Litter, engineer in charge, and Mr. W. J. Potter, superintendent. The castiron cylinders were furnished by R. D. Wood & Co., and the structural steel was fabricated by the Pennsylvania Steel Co.

The London Water Supply is to be increased by a new 3,000,000,000-gal. reservoir, the construction of which has just been undertaken at Chingford, in the valley of the River Lea.

Hydro-Electric Development in Colorado.

The development of water powers in the Rocky Mountains presents various interesting problems, some of the details being very different from the developments in the Mississippi Valley and the Eastern States. These conditions have been outlined in a paper by Mr. Lewis E. Ashbaugh, office engineer of the Central Colorado Power Co., prepared for the year book of the Engineers' Society of the University of Minnesota, from which the following notes have been taken.

As Colorado occupies the crest of the continent, the streams rise within its borders and flow in the various directions from comparatively small water sheds. The streams of the eastern slope emerge from the mountains on to the plains with small volume, but their great fall permits some interesting developments, while the Grand River of the western slope gathers the waters from tributaries to such an extent that a fall of 100 ft. through a canyon may be profitable to develop. On the eastern slope of Pike's Peak, within view of Colorado Springs, a 19-in. pipe line carries the water from a very limited watershed and delivers it to a power house at Manitou, near the cogwheel depot, under a head of 2,300 ft., and generating 3,000 h.p. In the vicinity of Boulder, just north of Denver, the continental divide, with elevations over 14,000 ft., approaches within 20 miles of the plains to the east, at 6,000 ft. elevation, again emphasizing great fall and small watershed.

The large use of the waters of the eastern slope for irrigation, and the early priorities giving ownership of these waters, renders difficult the utilization of some of the best streams for power purposes. The stream gaugings have, in the past, been largely made in order to secure information for irrigation requirements. Little record has been made of the winter flow. The streams discharge the larger part of their total annual flow during the summer months, the maximum flow usually occurring about the middle of June. This is the time of year when the irrigation needs are also at a maximum. During the winter the flow decreases so that power development using the low discharge would be impracticable. Storage reservoirs are therefore necessary to regulate and equalize the flow, and these must be located up in the mountains above the proposed intake. If the summer flow is not fully appropriated and beneficially used for irrigation, there may be enough water to supply both the power-plant and the irrigation interests, assuming that the power development does not include reservoirs sufficient to store all the discharge waters.

The order of procedure for using the stream discharge for power purposes is to store, during the summer months, all surplus water for which reservoirs may be found, using such part of the natural flow as may be required to operate the plant. As the discharge diminishes in the fall, stored water must be released to supply the plant, and such must continue until spring. A stream which discharges over 500 cu. ft. per second during all of June may dwindle to less than 5 cu. ft. per second during January, with an average of 70 cu. ft. per second during the whole year. The total discharge may exceed 50,000 acre-feet during June and July, and be less than 1,000 acre-feet during two or three winter months. With sufficient reservoir capacity in the headwaters, and with no irrigation rights to demand more than 70 cu. ft. per second, a development could be made to use an average flow of that amount, less evaporation and other small losses. But a single year's condition is not sufficient. A series of years must be considered, and the minimum possible flow taken as the only basis which will be satisfactory to the investing financiers and to purchasers of power. Again, the small reservoir capacity available may permit a regulation to only a small portion of the average for the year. If irrigation rights must be satisfied, it may be necessary to construct other reservoirs below the plant and above the irrigated fields, giving secondary storage, such that the regulated flow may be discharged into such reservoirs during the winter season, and thus be available for use on the fields during the summer.

The determination of the regulated discharge of the stream is an interesting problem, dependent on many factors, as total discharge of the year and distribution thereof, storage available, irrigation requirements, and so forth. The records of gaugings as made by the hydrographer are of great interest; the "Water Supply and Irrigation Papers" of the United States Geological Survey are in constant use. On large developments the company employs a hydrographer to make special investigations. Automatic register gauges are used on streams where detail data are required. Co-operative work with the United States Geological Survey is undertaken to the advantage of both parties.

Illustrative of the variable flow of mountain streams, a page of gauge-register curves (combined from as many sheets), is shown in an accompanying diagram. Curves 1, 2 and 4 are from Station A, up close to the Continental Divide, with its snowy peaks, above which station the drainage area is about 30 square miles. Curves 3, 5 and 6 are from Station B, on the same stream, but down near the plains, 13 miles from "A," with drainage area about 100 square miles. These curves show gauge heights for one week each. Coincident in time are curves 2 and 3, also 4 and 5.

These curves are interesting in showing the large daily variations, and also the decrease from summer to winter discharge, though neither the maximum summer nor minimum winter flow can be shown, as record sheets are not available. For the week shown by Curve 1, it is noted that on Sunday evening, at 8 o'clock, the discharge was 348 cu. ft. per second. At 2 o'clock on Monday afternoon the discharge was 233 cu. ft. per second, and at 1 o'clock on Wednesday it was only 210 cu. ft. per second. On Friday evening it had increased to 353 cu. ft. per second. The peak discharges occurred between 8 and 10 p. m., with low discharges soon after noon. During June the peaks occurred at about 5 p. m., while Curve 5, for Station B, shows the November peaks at 2 to 4 a. m., and Curve 6 shows December peaks at near noon of the following day. During June the snow line is down close to the stream, and the daily "high tide" or peak occurs soon after the sun has melted the snow, while later in the season the remaining snow is farther up on the mountains, and melts in less volume, thus requiring a longer time for the tide to travel down the valley to the gauging station.

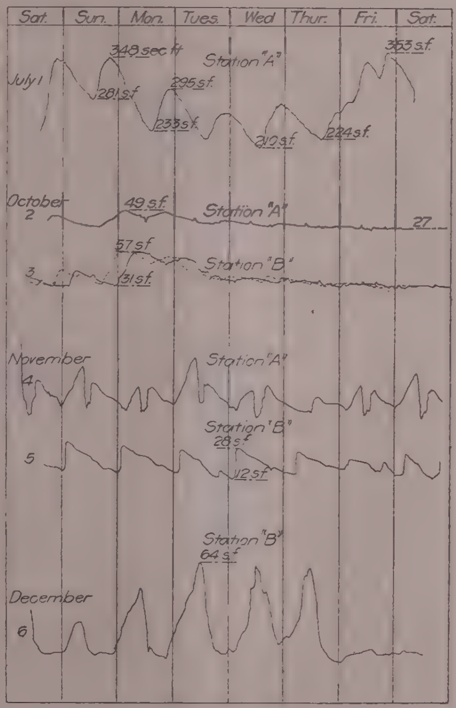
These curves show the value of automatic registers and continuous records. A single current-meter gauging, taken during the week represented by Curve 1, might be made at noon and get a very different result from the true average. The United States Geological Survey gauges are read by resident observers morning and evening, thus tending to get an average result for the day. Current-meter gaugings are then frequently made and corresponding gauge heights read, thus giving the rating table for estimating discharges for all gauge readings at the given station.

Curves 2 and 3, for October, show how the summer flow has decreased and the daily tide become less pronounced. If Curve 3 is shifted back 6 hr., shown by dotted lines, to allow for the distance between Stations A and B, the coincidence of maxima and minima becomes remarkable. The daily fluctuation is still large on some days. Curve 4 was taken when anchor ice was forming and causing fluctuations in the channel, but Curve 5, taken the same week at the lower station, shows pronounced and regular tide variations, at one time changing from 12 to 28 cu. ft. per second discharge in 2 hr. Curve 6 shows the effect of cold December weather. The variation is intensified in the record by the ice formations.

The annual record of a Colorado stream is shown by the discharge curve of the Grand River at Glenwood Springs, for the year 1904. During six winter months the discharge was less than 1,250 cu. ft. per second, while for three months it exceeded 3,000, and at one time reached 17,500 cu. ft. per second. In 1899 the maximum discharge reached 29,000 cu. ft. per second.

In the Eastern States much is said about the relation of run-off to rainfall, and the run-off per square mile. On some watersheds of the Rocky Mountains, the rainfall records show less precipitation than the run-off of the streams, due to there being much greater precipitation around the peaks than in the park and valley areas in lower altitudes where the towns are located and meteorological stations established. The effect of high altitude and proximity to snowy peaks is shown in comparative data obtained on two watersheds of nearly equal size of area about 25 square miles, separated by about five miles between borders, the one "fronting" for several miles on the Continental Divide, while the other does not quite reach up to the divide, and then with a pointed end as compared with the broad "front" of the first. The July, 1907, records show run-offs as follows: 11.25 cu. ft. per second per square mile for the first watershed, and only 1 cu. ft. per second for the other. The watershed which includes both of the areas just treated, together with an additional intermediate and tributary area of about equal size shows a run-off of 4.26 cu. ft. per second per square mile for the same period.

Considerable attention has here been given to hydrography because the water supply is the first essential, and Colorado mountain conditions are so very different from those of the Eastern States and the Mississippi Valley.



Records of Automatic Stream Gauges.

The Central Colorado Power Co. is planning some extensive developments, three of which are in process of construction, namely, the plants at Boulder, Gore Canyon and Glenwood. Investigation and filings have been made on several other projects, as Big Thompson and Roaring Fork. While plans are not definitely determined, and continued studies are revealing new methods and possibilities, brief general descriptions and explanations will be made of the Glenwood and Boulder projects. Sketch maps of these projects and of a portion of the State are given for reference. Figs. 2, 3, 4.

By means of such reservoirs as Grand Lake, Lehman and Williams Fork reservoir, it is planned to regulate the flow of the Grand River to 1,000 cu. ft. per second at Gore Canyon, and 1,250 cu. ft. per second at Glenwood. These are to be "base-load plants," to generate electricity at a continuous maximum rate. The fall of head at Gore Canyon is about 300 ft., or, with a proposed extension, about 400 ft. The development is very similar to the one at Glenwood. Any reservoirs above Gore Canyon do double duty in regulating the flow for the two plants, and it is possible that they will become still more valuable by the development of other powers on the Grand River both above and below Glenwood.

The Glenwood development is located in the beautiful canyon just above that city, along the Denver & Rio Grande R. R., the intake being near Tunnel No. 2 on that railroad. A diverting dam with flashboards or needles will form a pool and divert the water into the tunnel now being constructed. The waterway will be about 16 ft. wide and 10.5 ft. deep, with grade of 2.64 ft. per mile, to carry 1,250 cu. ft. per second. The length of tunnels will be about 12,000 ft. Two steel pressure pipes will take water from the tunnel forebay to the two turbine wheels and generators. The net head is about 165 ft., thus giving about 16,500 electrical horse power. As first planned, the present development would have only 9,000 ft. of tunnel, with 140 ft. head, and would look forward to an extension toward, and perhaps beyond the city of Glenwood, giving much increase in power. The tunnels are being bored through hard granite, requiring very little timbering and arch lining. The sides and bottom will be concreted and smoothly finished.

The Boulder plant is designed to carry the peak loads for the other base-load plants. Such reservoirs as Barker and Nederland will store the summer floods for winter use. A 36-in. concrete pipe line about 12 miles long follows along the hills at constant grade, cutting through some projecting ridges by tunnels and crossing deep ravines by inverted steel syphons, carrying a constant supply of water from Barker storage reservoir to Kossler equalizing reservoir. This pipe is a gravity waterway, and will carry about 54 cu. ft. per second at a maximum; this quantity is quite different from Grand River conditions. The Kossler reservoir is a terminal reservoir, receiving a continual supply from Barker reservoir, and transmitting to the power house as required by means of a steel pressure pipe over 9,000 ft. long, 56 in. diameter, by ¼ in. thick at the upper end, and 44 in. in diameter, and 1¾ in. thick at the lower end, where the head is 1,845 ft. During the late afternoon and the evening, and at other times of peak load, this heavy steel pipe will deliver to the power house the water which the gravity pipe line has been supplying to the terminal reservoir during the 24 hr. This terminal reservoir thus gives the value to the development; two impulse wheels will be used, the capacity of each being 9,000 h.p. Other interesting features of the Boulder development are the big reservoir dams, a 165-ft. masonry dam being planned for Barker, and one or two large earth dams for Nederland. Another development is possible by taking the water from the power house and conducting it by tunnels and pipe lines to the edge of the foothills, on the west side of the city of Boulder, a distance of about three miles, and affording a fall of about 400 ft.

The main transmission line will extend from Glenwood to Denver, passing through the large mining camps at Leadville, Breckenridge, Dillon, Georgetown and Idaho Springs. To this line will be connected the lines from Gore Canyon and Boulder. Branches will supply the manufacturing districts at Pueblo and the mines at Cripple Creek.

The Removal of Unprotected Facing-Point Switches has been carried out by the Pennsylvania R. R. on all of its lines east of Pittsburg and Erie, Pa., 1,078 of such switches having been removed in the past four years. Of these, only 125 have been replaced by trailing switches, the remainder having been abolished. During this time, in which over 500 miles of running tracks have been added and 409 new siding connections made on the lines east, 1,094 facing-point switches protected by interlocking have been put in.

Steel Sheet Piling Costs.

The cost of 140 26-ft. steel sheet piles driven to an average penetration of 22 ft. on the North Platte irrigation project is itemized as follows in the "Reclamation Record." The United States type of piling was used, which cost at the factory 70 cents per linear foot, and as its weight is 35 pounds per running foot, the cost therefore was 2 cents per pound. The freight rate from the factory at Pittsburg to Whalen, Wyo., was $1 per hundred pounds, thus making the total cost f. o. b. cars at Whalen, about $1.05 per linear foot.

The line of piles under consideration was driven in August, 1907, and forms a part of the south side of the cofferdam used in the construction of the concrete diversion dam on the North Platte River, at the head of the Interstate Canal. None of the piles were driven under water, and the material into which they penetrated consists of sand and coarse gravel. The piles were dragged from the railroad siding to the river bank, and carried across the river on cables. The pile driver outfit used was a Lidgerwood single drum 20-h.p. hoisting engine and a 2,800 lb. hammer, having an average drop of 8 ft. When no hindrance occurred by accidents to the machinery, the average number of piles driven per twelve hours was twenty-seven, with an exceptionally high run on August 9 of twenty-nine.

The regular pile driving crew consisted of one foreman, one engineer, and four laborers. Each of these men received 35 cents an hour for his work except in transporting the piles from the railroad station to the driver, in which case the laborers were paid for at the rate of 25 cents per hour and teams at the rate of 20 cents per hour. The total labor cost of unloading and moving the piles from the railroad to the driver was $53.25, making a unit cost per linear foot of pile of $0.015. The total labor cost for driving was $190.05, making a unit cost of $0.052 per linear foot of pile driven.

Below are tabulated the total and unit costs of the piles in place distributed under the headings of plant depreciation, labor, materials and supplies. The depreciation on the engine was about 2 per cent. of its original cost, while that on the driver was about 30 per cent. of its original cost, including repairs made on it. The charge for materials contains in addition to the piling and freight thereon, $28 worth of wood fillers used in connection with the piling. The charges under supplies consist of coal and oil for the engine, and labor for carrying drinking water. Six tons of coal were used, at $5.50 per ton.

Distribution of Costs	Total cost.	Cost per foot of pile.
Plant depreciation	$60.00	$0.016
Labor	243.30	0.067
Materials	3,850.00	1.058
Supplies	44.18	0.012
Total	$4,197.48	$1.153

FORESTRY WORK by railways in this country was begun years ago, but was unsuccessful for one reason or another, and forest planting on a well-arranged plan has only been carried on by those companies since 1900. Practically all the trees were black locust and catalpa until recently, but the Pennsylvania R. R. has lately been setting out red and pin oak, larch and chestnut; and the Santa Fé has approved a plan of planting 7,000 acres of eucalyptus. Forestry specialists seem to be agreed that the setting out of plantations will not be profitable to railways as a means of producing cross-ties exclusively, on account of its high cost, but will prove useful as a means of improving existing timber lands which can be acquired at reasonable prices. The purchase of cut-over lands at low prices and their rehabilitation under the direction of experts is regarded as the best policy to be followed.

Air Leakage in Steam Condensers.

A plea for scientific consideration of the subject of air leakage in steam condensers was made in a paper by Mr. Thomas C. McBride, of Philadelphia, Pa., before the American Society of Mechanical Engineers at the recent spring meeting. Although experience has shown that the presence of a small amount of air in a steam condenser has a marked effect upon its capacity and efficiency, there has apparently, Mr. McBride states, been no serious attempt made to determine quantitatively the amount of air mixed with steam in ordinary practice, nor to definitely define or limit this amount to exact figures in condenser contracts and guarantees.

Mr. McBride believes and assumes that the amounts of air passing through different condensers may be best measured and compared on the basis of the relation of the volume of this air to the volume of the steam being condensed, both being at the pressure and temperature existing in the particular exhaust pipe. The most convenient unit of volume is the volume of the water of condensation coming from the steam. The ratio of the volume of air passing through the air pump to the volume of water of condensation is then used to measure the air.

The most reliable index of the proportion of air to exhaust steam customary in existing plants he finds to be in the ratings of air pump capacity in general use. He states that manufacturers of vertical twin air pumps and surface condensers usually guarantee them for 26 in. vacuum (30 in. barometer) and 110° Fahr., hot well temperature, with 70° injection and 110° discharge temperature, under which conditions it is customary to furnish an air pump of a displacement of about thirteen times the volume of the water condensed. This presumes the presence of 4.0 volumes of air per 10,000 volumes of exhaust steam as the upper limit to be anticipated in American practice. Horizontal air pumps for 26 in. vacuum and 110° hot well temperature are rated by most manufacturers as having a displacement of about twenty times the volume of the water of condensation. This rating presumes the presence of air in the proportion of 6.3 volumes to 10,000 volumes of exhaust steam. Air pumps for jet condensers for 70° injection and 26 in. vacuum, on the presumption that the discharge temperature will be 110°, are usually rated at a displacement of 52 times the volume of the water condensation. On these presumptions, this displacement will be made up of:

Condensed steam	1.0	Volumes
Air entering with steam through boiler	0.3	"
Condensing water	26.0	"
Air entering with condensed water	10.5	"
Air entering by leakage	14.2	"
Total	52.0	"

The 14.2 volumes of air in the air pump suction at 4 in. absolute and 110° were before condensation 4.7 volumes per 10,000 volumes of exhaust steam, or to refer to the basis used before for the surface condenser, the air entering by leakage and in the boiler feed total 14.5 volumes and correspond to 4.8 volumes per 10,000 volumes of exhaust steam.

Mr. McBride, therefore, considers it safe to say that a provision to handle about 4.5 volumes of air per 10,000 volumes of exhaust steam at 26 in. is ample in steam engine practice, and that more air than this indicates excessive leakage, this depending, however, somewhat on the size and type of plant. Of the air pump capacity presumed necessary by manufacturers, about 7 to 9 per cent. is thus required to handle the air coming into the condenser with the feed water and the balance is required to remove the air entering by leakage and is largely an unnecessary expenditure. The responsibility for limiting the amount of air is, he states, with the prime mover and its piping, and it is only reasonable to expect those controlling these features of the plant to indicate the amount of air the air pump must handle and not to compel the condenser manufacturer to guess at it.

It must be evident that comparisons of results obtained in different plants and from different types of condensers cannot possibly be trustworthy unless the proportion of air for each case has been determined and due allowance made therefor. The amount of air present exerts a great influence on (a) the relation between temperature and vacuum in the condenser; (b) on the air removing capacity of the air pump; and (c) on the heat transferring capacity of the surface or spray. The plant so fortunate as to have the smallest air leakage might have an advantage sufficient to wipe out entirely the benefits that might be gained by any refinements in the design of a condenser under comparison. Thus, a comparison between a condenser provided with a cooler, and a unit which had its cooler removed, shows results apparently favoring the latter. The methods of measuring and comparing air leakages suggested in this paper would not only show this to be an erroneous conclusion, and that air cooling was of great benefit to the unit, but it would also show the amount to be gained by such cooling.

The suggested measurement of the air coming to condensers necessitates the following readings:

1. Pressure in air pump suction.
2. Temperature of air pump suction.
3. Quantity of steam being condensed.
4. Effective vacuum pump displacement.

For ordinary work and with properly built condensers, the vacuum in the condenser could be taken for the first reading, but for exact work the pressure and temperature should both be taken at the same point in the suction pipe. The third quantity is generally well known. The fourth quantity requires readings of the speed of the vacuum pump and an estimate of its volumetric efficiency from its indicator card.

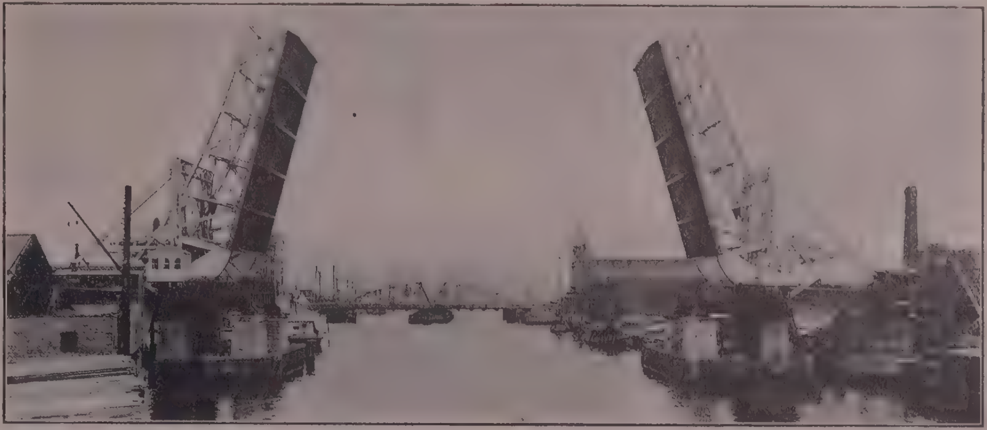
For barometric condensers and jet condensers with air pumps, certain allowances have to be made, requiring for their determination some previous practice with condensers of other types.

As the data necessary for the estimation of air in condensers is so easily obtained, it is hoped that this paper will eventually result in the application of the same scientific methods to air leakage in condensers that are now applied to all the other features of the power house. A full appreciation of the effect of air in condensers on coal consumption and an exact knowledge of the amount and probable source of this air should result in generally higher vacuums and an increase both in the efficiency and capacity of the plant. The methods suggested in this paper can be used in the analysis of existing condenser plants to see how their results can be improved, assisting in determining whether the cooling features or the air removing features are limiting the vacuum, and if the latter, determining just what are the improvements that could be made and how much.

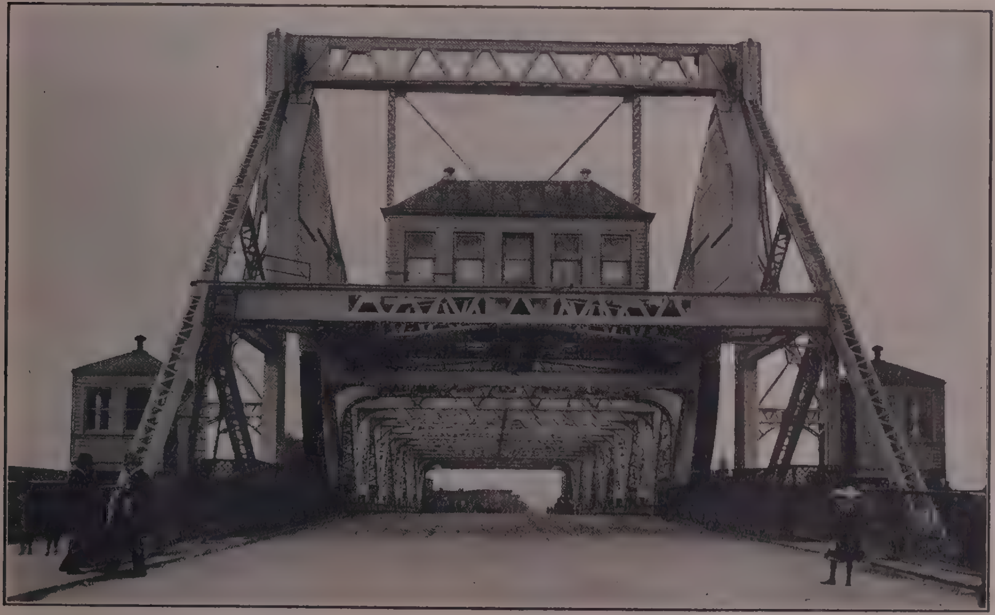
SEWER TUNNELS about three miles long have been driven in Hamburg, Germany. A 9-ft. 10-in. circular tunnel was driven with a shield without air pressure at about $81 per foot, a 7-ft. 10-in. tunnel was driven by ordinary methods without air pressure at $31.60 per foot and with air pressure at $49.50 per foot. A 7-ft. 10-in. tunnel was driven with a shield and air pressure at $66.50 per foot. The tunnels were lined with brickwork instead of cast-iron segments. Great difficulty was experienced with the air pressure, for if it was sufficient to permit work in the upper chamber of the shield, the water rose in the lower chamber, and when extra pressure was applied to keep the lower chamber dry, the air immediately escaped upward through the soil.

Vernon Avenue Bascule Bridge, New York City.

The increasing requirements of the very heavy navigation along Newtown Creek between the Boroughs of Brooklyn and Queens, New York City, induced the vessel and towing interests operating along this busy stream to urge more than ten years ago the removal of the obstructive center pier swing bridge spanning the creek near its mouth and connecting Vernon Ave., Queens, with Manhattan Ave., Brooklyn. This bridge crossed the channel at a level so low as to necessitate an opening of the bridge for every passing craft, resulting in delays expensive to both navigation and the heavy highway traffic over the bridge. The necessity for relief at this point was appreciated by the United States engineer then in charge of the improvements of New York Harbor, Col. G. L. Gillespie, U. S. A., who recommended condemning the old bridge and replacing it with a structure more suited to the needs of the increasing navigation and street traffic.



The Vernon Avenue Bridge and a Neighboring Swing Bridge.



View Showing Location of Tail Latches on Top Braces.

Following proper legislation on the subject, a joint committee was appointed by the counties of Kings and Queens in 1896 to consider the best means of carrying out the project. The committee decided to advertise for competitive designs and this was done, the advertisement calling for drawings showing the general design of the proposed structure, length of movable span and approaches, width of clear waterways, width of roadway and sidewalks, dimensions of all parts, loads to be sustained and strain sheets showing the forms and construction of typical members. Each design was to be made in accordance with the regulations of the United States War Department and was to be accompanied by a typewritten description of the proposed structure, giving the motive power to be used and its manner of application in the operation of the bridge. A number of designs were submitted in accordance with this advertisement, but nothing definite was done in the selection of a design for several years, although various phases of the subject were considered by the Department of Bridges. The original project of a low-level swing bridge was abandoned, as the increasing navigation along Newtown Creek would necessitate almost constant operation of the draw span and consequent delay to the heavy traffic over the bridge. The low-level bridge was also objectionable because of the danger both to the bridge structure itself and passing vessels on account of the strong current in the channel caused by the tide. Vessel interests urged a bridge which would have sufficient clear height underneath to allow a large percentage of the water traffic to pass without necessitating a movement of the bridge. Such a height would also permit a viaduct approach to the bridge on the north to clear the tracks of the Long Island R. R., which cross Vernon Ave. in the Borough of Queens about 400 ft. from the bridge.

In 1901, the then Commissioner of Bridges, Mr. John L. Shea, after an exhaustive investigation of the merits of the various types of bridges proposed, and following the recommendations of the chief engineer of the department at that time, Mr. Samuel R. Probasco, adopted a double-leaf through Scherzer rolling lift bridge as best fulfilling the requirements of both the highway traffic and navigation, and being also the most economical of any of the designs considered. The Department of Bridges accordingly contracted with the Scherzer Rolling Lift Bridge Co., Chicago, for complete detail plans and specifications which were placed on file at the office of the Commissioner and bids for construction called for on Dec. 5, 1901. The contract was awarded and construction carried through to completion, the bridge being formally opened by Mr. George E. Best, Commissioner of Bridges, and Mr. O. F. Nichols, chief engineer, on Oct. 17, 1905. Arrangements were made by the Department of Bridges to provide for the highway traffic during the construction of the new bridge by erecting a bridge at the adjacent street upstream, Oakland St. This bridge was opened to traffic on Dec. 26, 1902, and remained in active use until the opening of the new bridge.

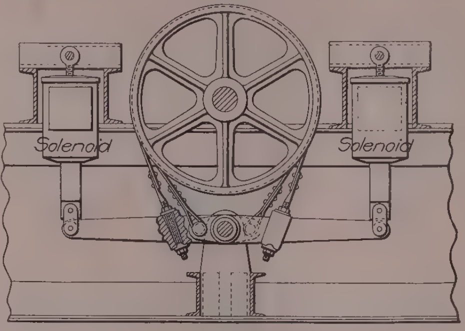
The importance of the structure as a means of communication between two busy districts was emphasized in the formal opening of the bridge to highway traffic, the occasion being celebrated as a holiday by the neighboring business interests. The importance, too, of that part of Newtown Creek as a commercial waterway is shown by the record of operation of the new bridge for the year 1906. In the twelve months, from January to December, of that year the bridge was operated for the passage of vessels on an average of 1,391 times per month, the maximum number of operations being 1,584 in the month of October. The bridge was operated 72 times in one day in the month of December. During this same year 53,178 vessels passed through the opened bridge while 44,882 vessels passed under the bridge without necessitating the opening of the draw-span. The saving in time to the heavy highway traffic through the use of the new bridge is shown in the fact that the delay to street traffic since the opening of the new structure is less than one-half of the delay caused by the adjacent bridge at Oakland St., in use during the construction of the new bridge.

This entire structure, including masonry and steel approaches, has a length of 1,699 ft., of which 361 ft. are in the masonry approaches, 207 ft. on the north end beginning at Fourth St., and 154 ft. on the south end at the intersection of Ash St. and Manhattan Ave. The viaduct approaches on both north and south sides are of steel supported on steel bents resting on concrete footings, the north viaduct having a length of 826 ft. and that on the south a length of 180 ft. The movable span and the two adjacent fixed spans have a total length of steel work of 332 ft., the movable span, center to center of bearings, being 172 ft. with a single roadway of 40 ft. clear width and two sidewalks each 8 ft. wide. The bridge provides a clear waterway for navigation 150 ft. wide, measured between protection fenders at right angles to the center line of the channel. The clear height from mean high water to the under side of the floor beams is 24 ft. The clear roadway of this bridge between trusses is the widest of any through type of bascule bridge yet constructed.

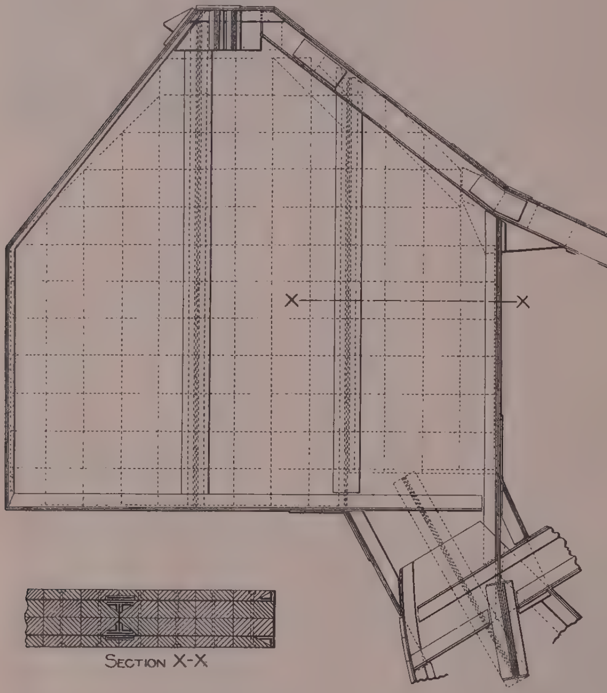
The movable span is a double-leaf through Scherzer rolling lift bridge, each leaf consisting of two riveted cantilever trusses. The top chord of each truss is carried downward in a sweeping curve from the anchorage columns to a minimum clearance over the roadway of 15 ft., making, with the straight rear end members, a pleasing and artistic outline for a through type of draw span.

The bridge is operated by electric power, two motors of 40 h.-p. each being provided for each leaf. In the operation of the bridge the power is applied to each of the movable leaves by two operating struts moving horizontally inside of the trusses, the machinery driving the operating struts being placed on stationary platforms over the fixed spans at the rear end of each of the moving leaves. The electric equipment is designed to permit of each leaf being operated separately from its side of the channel, and by means of submarine cables the control of both leaves is located on the Queens Borough side of the channel. Indicators are placed so as to be seen from each operator's house showing the various positions of each leaf of the bridge during operation, both day and night.

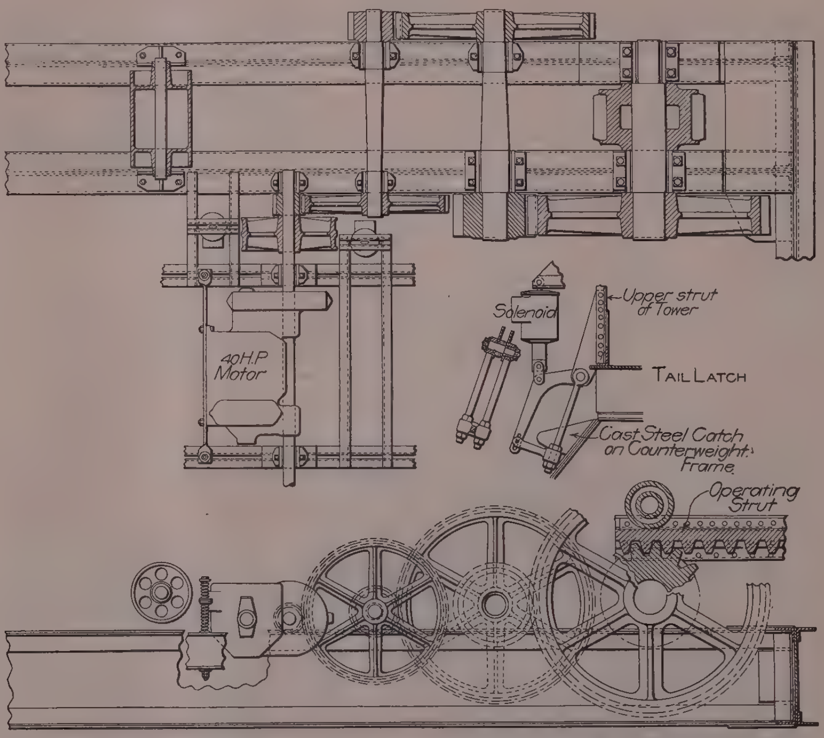
The main supporting piers, both front and rear, are of concrete faced from 2 ft. below mean low-water level to the under side of the coping with ashlar limestone masonry, the coping and bridge seats being of granite. Each pier has a batter of ½ in. to the foot above the footings, the granite coping course extending 18 in. beyond the face of the top course of limestone. The front or river piers rest on pneumatic caissons carried to rock. The rear piers in which the anchorage columns are imbedded rest on piling cut off at 18 ft. below high water. These piers were built in open caisson work, the toe of the concrete being 5 ft. below pile cut-off. The bridge has been remarkably successful in more than two years of service, having caused no delay to traffic on the highway or in the river.



Power Brake for Bascule Bridge.



Counterweight Frame with Tail Catch.



Operating Mechanism and Tail Latch, Vernon Avenue Bascule Bridge.

The entire construction was executed in charge of the Department of Bridges of the City of New York, Mr. E. A. Byrne, engineer in charge. The approaches and substructure were designed by the Department of Bridges. The movable span superstructure, operating machinery and electrical equipment were designed by The Scherzer Rolling Lift Bridge Co., Chicago, Mr. Albert H. Scherzer, president, and were constructed under the consulting engineering supervision of that company. The contract for the entire work, both substructure and superstructure of the movable bridge and approaches, amounting to $567,082.46, was executed by the United Engineering & Contracting Co., New York City, the superstructure being fabricated by the Elmira plant of the American Bridge Co.

The Cement Mill operated by the United States Reclamation Service at Roosevelt turned out 12,016 bbl. of cement during March, which is stated in the "Reclamation Record" to have been manufactured at a unit cost of $1.84. The sand-crushing plant at the same place furnished 2,268 cu. yd. at a unit cost of $1.27.

Permeability Tests of Concrete with the Addition of Hydrated Lime.

A paper read before the American Society for Testing Materials, by Sanford E. Thompson, M. Am. Soc. C. E., Consulting Engineer, Newton Highlands, Mass.

These permeability tests were made to determine the effect of adding various percentages of hydrated lime to concrete of different proportions. The specimens were similar in a general way to those designed by the writer for previous tests, as described in the "Proceedings" of the American Society for Testing Materials, Vol. VI, p. 373, but were of much larger size; in fact, they were considerably larger than have hitherto been customary in permeability tests. The blocks of concrete were 20 in. in diameter by 16 in. thick, and the water was introduced into the center of the block as described and illustrated in this paper.

The proportions of concrete selected were those which are customarily used in concrete construction, except that a slight excess of sand was taken in proportion to the stone to insure homogeneous specimens, thus avoiding the possibility of stone pockets, which are more liable to occur in small-size specimens than in practical construction.

The materials, which are more fully described later in the paper, are quite representative, so that the results obtained are applicable to usual conditions in construction work. The cement was Lehigh brand; the crushed stone was conglomerate rock, which resembles trap in its characteristics and tests: the sand was a good, coarse bank sand. The hydrated lime was Pine Cone brand made by the Rockland-Rockport Lime Co., of Rockland, Maine.

The percentages of hydrated lime to add were selected after a careful study of the materials and the proportions, the plan being to use larger percentages of lime with the leaner concrete mixtures because these contained more voids to be filled. The proportions of concrete and the percentages of lime selected by this preliminary study were as follows:

Proportions of concrete.	Hydrated lime.	Proportions of concrete.	Hydrated lime.	Proportions of concrete.	Hydrated lime.
1 : 2 : 4	0	1 : 2½ : 4½	0	1 : 3 : 5	0
"	4%	"	6%	"	8%
"	7%	"	10%	"	14%
"	10%	"	14%	"	20%

The tests were laid out by the writer at the request of the Rockland-Rockport Lime Co., and were made by the Boston Elevated Railway Co. in their Cambridge laboratory. The scheme of tests was approved by Mr. W. E. Healey, of the Rockland-Rockport Co., and by Mr. George A. Kimball, chief engineer of the Boston Elevated Railway Co., and the results are here presented with their permission.

Conclusions.—The following conclusions are drawn as the result of the experiments:

1. Hydrated lime increases the water-tightness of concrete.

2. Effective proportions of hydrated lime for water-tight concrete are as follows:

Port. cem.	Sand.	Stone.	Hyd. lime.
1 part	2 parts	4 parts	8%
1 "	2½ "	4½ "	12%
1 "	3 "	5 "	16%

These percentages are based on the weight of the dry hydrated lime to the weight of the dry Portland cement.

3. The cost of large water-proof concrete structures frequently may be reduced by employing leaner proportions with hydrated lime admixtures, and small structures, such as tanks, may be made more water-tight.

4. Lime paste made from a given weight of hydrated lime occupied about 2¼ times the bulk of paste made from the same weight of Portland cement, and is therefore very efficient in void filling.

Although the character of the sand and stone used in the concrete will affect the best percentage of lime to use, the present materials are representative of average materials throughout the country, so that the results should be of general application. Coarser sand would naturally require slightly larger percentages of lime, and finer sand, that is, sands having a larger percentage of fine grains which pass a sieve with 40 meshes to the linear inch, would be apt to require less lime, since sands containing considerable fine material produce a more water-tight although a weaker concrete.

A study of the curves which are given later in the paper suggests that for coarser sands than used in the experiments, or in cases where the water pressure must be applied in one month or less after laying, the percentages given above may be increased by about 25 per cent., while for many classes of work 25 per cent. less than the above percentages will be satisfactory.

For mortar in proportions corresponding to the cement and sand in the different concretes, the same percentage of hydrated lime would naturally be selected; thus 1 : 2 mortar corresponds to 1 : 2 : 4 concrete, 1 : 2½ mortar to 1 : 2½ : 4½ concrete, 1 : 3 mortar to 1 : 3 : 5 concrete.

Increasing the proportions of stone to, say, 1 : 2 : 4½, 1 : 2½ : 5, or 1 : 3 : 5½, should have no effect on the percentage of hydrated lime to be selected, but for water-tight concrete it is always advisable to have a slight excess of mortar to avoid stone pockets.

Results of Experiments.—The specimens ready for testing are shown in the photograph, Fig. 1.

The water is introduced into the middle of the concrete cylinders. As it enters the specimen the pipe is enlarged so that an area of 12½ sq. in., that is, a circle 4 in. in diameter, is exposed to the water, and the water must flow through 8 in. of concrete.

The accompanying table gives the results of the tests in detail, while the curves in Fig. 2 also present the effect of the hydrated lime upon the water-tightness of the concrete.

The percentages of hydrated lime are based on the weight of the cement, these being added to the cement and not replacing it. The variation in the ages of the specimens in different proportions slightly affects the results and accounts in part for the fact that the 1 : 3 : 5 mixtures are nearly as water-tight as the richer proportions. Specimen No. 5, which shows practically no flow, is evidently erratic.

PERMEABILITY OR FLOW OF WATER THROUGH CONCRETE 8 IN. THICK.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
		Flow of water under 7-ft. head.			Flow under pressure of 60 lb. per sq. in.			
No.	Hydrated lime per cent.	Age, days.	Duration of measured flow, hours.	Flow, grams per hour.	Age, days.	Pressure applied before measurement, hours.	Duration of measured flow, hours.	Flow, grams per hour.
1 : 2 : 4 *Concrete.*								
1	0	18	161	2.7	40	24	4¼	74.8
2	4	18	161	1.2	41	18	5	28.4
3	7	18	161	1.0	42	18	6¾	5.2
4	10	15	161	1.0	46	6	18	1.6
1 : 2½ : 4½ *Concrete.*								
5	0	30	169	0.3	44	17	6	1.1
6	5	30	169	1.9	45	18	6	32.5
7	10	29	169	0.8	49	..	21	0
8	14	29	169	0.7	50	..	27	0
1 : 3 : 5 *Concrete.*								
9	0	26	169	9.8	50	6	14	70.6
10	8	26	169	1.1	51	8	17	3.6
11	14	28	169	1.1	50	28	13	10.7
11	20	28	169	1.2	53	9	15	0.7

The specimens were first subjected to a head of 7 ft. by filling the vertical pipes with water, and while the results of these preliminary tests are of small value because the pressure in most cases was not enough to force the water through the specimen, the quantity being measured before entering the concrete, the results are given in the table, column 5, as confirming in a general way the tests of the concrete under city pressure. The tests under a 7-ft. head, column 4, were maintained for several days, and the flow was recorded each day, the average only being given in column 5.

After a longer period of setting, varying from 40 to 53 days, the specimens were connected with the city water pressure, which averaged 55 to 60 lb. per square inch. Before measuring the flow, the water was continued for several hours (column 7) to equalize the flow, and then the flow was measured for a period of several hours and recorded in grams per hour.

The flow given in grams per hour is very nearly the same as the flow expressed in ounces per 24 hours. The curves in Fig. 2 are plotted from the table.

The result from one specimen, the 1 :2½ :4½ mix without any hydrated lime, is erratic, and this test is omitted in making up the curves. All of the tests show some variations in the lower portions of the curves, but when the flow through the specimen is as small as 10 grams per hour, which is approximately equal to about 8 ounces or ½ pint per 24 hours, the concrete is so nearly watertight that slight variations are unavoidable. The curves are based on a study of all of the tests. The pressure, nearly 60 lb. per square inch, which is equal to a head of about 140 ft., is unusual in engineering construction. Previous tests, by Fuller & Thompson, Trans. American Society of Civil Engineers, Vol. LIX, p. 133, have shown that the flow through concrete is almost directly proportional to the pressure, so that for lower pressure the flow will be correspondingly less. The thickness of the concrete was only 8 in. and watertightness increases greatly with the thickness.

Fig. 1.—Permeability Test Blocks.

In selecting the best percentages of hydrated lime for the different proportions of concrete, these points, as well as the fact that the 1 :3 :5 specimens are slightly older than the others, have been taken into account, and as a result of this study proportions suggested for general practice are 8 per cent. for 1 :2 :4, 12 per cent. for 1 :2½ :4½, and 16 per cent. for 1 :3 :5 concrete, these percentages of lime being based on the weight of the cement. As stated, smaller percentages than these, for example, 6 per cent. for 1 :2 :4, 9 per cent. for 1 :2½ :4½, and 12 per cent. for 1 :3 :5, give results which are satisfactory for many conditions in practice, while in certain cases a larger percentage may be chosen.

Frequently it is advisable to make special studies or tests with the materials available to determine the best percentage.

The sand was ordinary New England bank sand and the crushed stone was conglomerate or pudding stone, whose quality and general characteristics for concrete are similar to trap.

The mechanical analyses of the aggregates are given in the accompanying table, the sizes of grains being given as the total percentage passing the various sieves.

Making the Specimens.—The apparatus was designed before making the specimens, and full directions covering the details of the measuring and mixing of the materials and the making of the specimens were prepared in writing and sent to the laboratory of the Boston Elevated Railway Company, the tests being made by Mr. F. B. Edwards, assistant engineer. A working drawing of the specimen, as molded with the apparatus required, is shown in Fig. 3. Instead of using a wooden mold, the sides were formed by two pieces of wrought iron bent to half-circles and bolted together. These rested on a plank platform to form the bottom of the mold until the concrete had set.

MECHANICAL ANALYSES OF THE AGGREGATES.

No. of sieve.	Width of mesh of sieve.	Total percentage passing.	
		Sand.	Stone.
2 in.	2.00 in.	..	100
1½ in.	1.50 in.	..	92
1 in.	1.00 in.	..	76.8
½ in.	0.50 in.	..	48.8
¼ in.	0.25 in.	..	12.6
No. 5	0.16 in.	97	1.8
No. 12	0.0583 in.	75.8	1.8
No. 20	0.0335 in.	59.7	1.3
No. 40	0.0148 in.	25.7	1.1
No. 100	0.0055 in.	4.0	0.7
No. 200	0.0030 in.	1.8	0.4

The proportions were based on volume measurements, assuming 100 lb. cement to the cubic foot, but for greater accuracy in making the tests the weights per cubic foot of the sand and stone were determined and the measurements made by weight. The cement and hydrated lime were first thoroughly mixed to give a uniform color, then the sand was added and mixed dry. The stone was finally added and sufficient water used to make a soft, mushy mix, such as would be adopted in construction. The mixing was very thorough, but no more so than would be expected in practice with ordinary mixers.

Since previous tests by the writer showed that the top of a block as molded is much more watertight than the bottom, because of the fine material which rises to the surface, the specimens were made in an inverted position, so that Fig. 3 shows the specimen upside down when compared with its position in the permeability test. The iron pipe fittings were placed upright in the molds, as shown, fittings which were easily obtainable being selected to cheapen the cost. These were so placed as to give a thickness of 8 in. of concrete in all directions, and a surface for the water to strike against of 12½ sq. in., that is, a circle 4 in. in diameter. The fittings were filled with sand before placing the concrete and the top covered with oiled paper to prevent the concrete getting into the pipe.

The bottom of the mold was covered with a thin layer about ½ in. thick (instead of 1 in., as given in the drawing) of neat cement, and neat cement was also plastered around the fittings which had first been thoroughly cleaned and scraped with a brush, thus forming a perfectly tight joint. There was no leakage in any of the specimens, in fact, it would appear that it is unnecessary to insert the pipe so far into the specimen, 2 or 3 in. probably being sufficient.

When the concrete had hardened, the sand and oiled paper were removed from the pipe and the surface chipped free from mortar down to the clear concrete, the large size of the fittings permitting a chisel to be readily worked. The top of the specimen as molded was also chipped down so that the stones were exposed in the surface of the concrete. When ready to test, each specimen was supported by 2 x 4-in. pieces of studding, which are shown in Fig. 1, and the fittings connected with a pipe passing up through the ceiling to the floor above. Before connecting, the fittings were partly filled with sand, thoroughly washed and screened to about 30-mesh size. A funnel 2 in. at the top reducing to the size of the neck of a large bottle was placed in each specimen to assist in collecting the water.

The concrete in each specimen weighed over 400 lb.

Proportioning the Hydrated Lime.—Few tests have been made to determine the effect of the introduction of special materials into concrete to increase its watertightness. The percentages of hydrated lime to use were determined by applying the conclusions derived from tests with graded concrete aggregates and the results of the tests indicate that mechanical analysis methods can be applied to the laying out of permeability tests.

For the 1:2:4 proportions the percentages of hydrated lime were selected partially by judgment. From previous tests by the writer, 2 per cent. of hydrated lime had been found to increase the watertightness of 1:2:4 concrete made with sand and gravel, and 4 per cent. of lime to still further increase it. The minimum percentage was therefore selected as 4 per cent., and the maximum percentage for the 1:2:4 mix was taken as 10 per cent., so as to confine the results to comparatively narrow limits. One intermediate percentage was also chosen, thus giving 0, 4, 7 and 10 per cent. for the 1:2:4 concrete.

Leaner concrete mixtures evidently require more fine material for watertightness, because there are more and larger voids. Furthermore, tests in general indicate that with a lean mix we can add much more fine material without reducing the strength, and that this addition up to a certain limit is beneficial not only for watertightness but also for strength.

For selecting percentages of lime for the 1:2½:4½ and the 1:3:5 concrete, which would compare with the percentages selected for the 1:2:4, methods of mechanical analysis were employed, the object being to obtain mixtures for each of these other proportions which would have substantially the same sizes of particles. In Fig. 4 the mechanical analysis of the cement, sand and stone are plotted and a combination curve of these three materials made for 1:2:4 and 1:3:5 concrete. The combined curves are simply the mechanical analyses of the mixture of the three dry materials in the given proportions. (For description of mechanical analysis methods by William B. Fuller, see Chapter XI, "Concrete, Plain and Reinforced"; also paper on "The Laws of Proportioning Concrete," by Fuller & Thompson, in Trans. American Society Civil Engineers, Vol. LIX, p. 67.)

The curves of the individual materials indicate the percentages passing the sieves of various sizes. For example, taking the curve for the broken stone, which is the lower curve, all the stone, or 100 per cent., passes a sieve 2 in. in diameter, 92 per cent. passes a 1½-in. sieve, 76.5 per cent. a 1-in. sieve, and so on. The sand and cement curves are plotted in a similar fashion.

Having plotted these individual curves, it is possible to calculate the percentages and plot combined curves showing mixtures of these three materials in any given proportions.

The making of these combined curves is quite simple. Thus, for example, to find the point on the 1:2:4 curve where it cuts the ordinate corresponding to the No. 12 sieve, the sums of the percentages of the individual materials at this same ordinate are taken in the proportion which they bear to the concrete mixture. All the cement is finer than the No. 12 sieve, and since the cement is one part of the seven parts in the mixture, one-seventh of 100 per cent. represents the percentage of cement in the mixture at the given ordinate. Similarly, since there are two parts of sand in the seven parts, the sand percentage at the No. 12 ordinate, 76 per cent., is multiplied by two-sevenths, and the stone percentage, 1.8 per cent., by four-sevenths, thus giving as the point on the No. 12 sieve ordinate in the combined curve:

1/7 x 100 per cent. = 14.3 per cent. = cement
2/7 x 76 per cent. = 21.7 per cent. = sand
4/7 x 1.8 per cent. = 1.0 per cent. = stone
37.0 per cent. = 1:2:4 mix

The other points in the curves are found in a similar manner.

Permeability depends more upon the gradation of the fine particles than of the coarse, and the finer material, that is, the left-hand portions of the curves, is therefore replotted in Fig. 5 to larger scale. Referring to the curves for 1:2:4 and 1:3:5 proportions, we notice that they are quite similar above, that is, to the right, of the No. 20 sieve, and that the greatest proportionate difference between them is below the No. 40 sieve. We may assume therefore that the particles lying below the No. 40 sieve are those which chiefly affect the watertightness. Therefore, the finer sieves, No. 100 and No. 200, must be chiefly used in studying the percentages of lime to add.

Several percentages were tried, and it was found that to make the 1:3:5 curve most nearly similar to the 1:2:4 concrete, 10 per cent. of lime must be added to the 1:3:5 mix. The curves of these two mixtures, together with the curves of concrete with no lime, are shown in Fig. 5. The other proportions were also plotted in the same way, but are omitted in the diagram for the sake of clearness.

The intermediate proportions for the 1:3:5 mix were thus determined upon as 8 and 14 per cent., while for the 1:2½:4½ proportions, 0, 6, 10 and 14 per cent. were found to correspond with the selected proportions for 1:2:4. Since a given weight of hydrated lime makes more than twice as much paste as the same weight of cement, the lime percentages are doubled when combining them with the other curves.

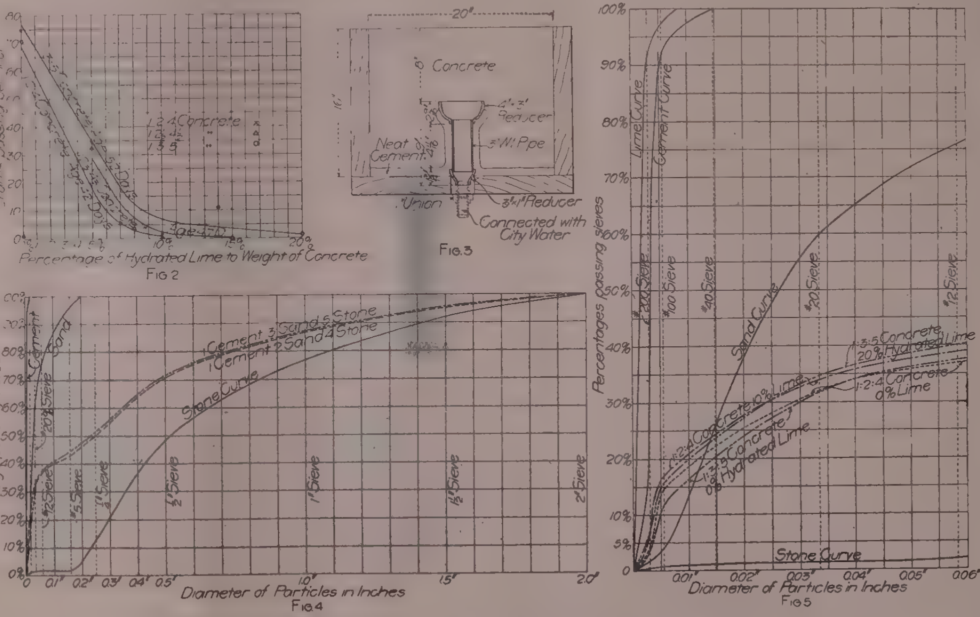


Fig 2

Fig. 3

Fig. 4

Fig. 5

Detail of Specimens, and Curves Showing Various Results of Tests.

Previous Tests with Hydrated Lime.—A few permeability tests with hydrated lime admixtures made by the writer in 1903 indicated it to be a valuable material for waterproofing. Later, in 1906, when the reservoir at Waltham, Mass. (see Engineering Record, Jan. 12, 1907, page 32), which is 100 ft. in diameter and 43 ft. high, was under consideration, the writer was consulted in the framing of the specifications and made another series of tests, using as follows:

Permeability Tests of 1:2:4 Concrete for Waltham, Mass., Reservoir, 1906.

Concrete 4 Inches Thick. Pressure 80 Lb. Per Sq. In.

Percentage of hydrated lime.	Flow in grams per min. (maximum flow) At 14 days.	At 21 days.	At 28 days.
0 per cent.	5.52	2.92	1.91
2 per cent.	9.20	2.55	1.63
4 per cent.	2.82	1.49	0.76

Percentage of lime is in terms of weight of cement.

As a result of these tests 5 per cent. of hydrated lime was adopted to mix with the 1:2:4 concrete in building its walls. The results were satisfactory, the only seepage occurring at joints formed between different day's work, where the bond between the old and new concrete was not made with sufficient care.

Construction of Water-Tight Concrete.—A paper on permeability is incomplete without reference to the general principles of construction, which must be followed to produce a water-tight concrete. The concrete materials may be perfectly graded and the proper proportions of cement and of hydrated lime used, and then if the concrete is poorly mixed, mixed with insufficient water, or improperly placed, or if joints are left in the mass, the wall will invariably leak.

In the first place, the mixture must be thorough, and in the second place, sufficient water must be employed to give at least a "mushy" mix, so that it will settle into place with only a small amount of ramming.

Fully as important as the mixture is the bonding of the concrete between two days' work. For a small structure which must be water-tight, it is advisable to place the concrete continuously, allowing no joint whatever, and not even permitting the concrete to stiffen up between the batches. Even an interruption of an hour in the middle of a hot day has been known to form a joint which will allow water to pass. If continuous work is impracticable, the old surface of the concrete must be thoroughly cleaned of all dirt and laitance or partially set cement, so as to expose the concrete. A layer of neat cement paste of soft consistency, or else of 1:1 mortar, should be then spread upon the old concrete after thoroughly wetting it, and the new concrete laid before this mortar has stiffened.

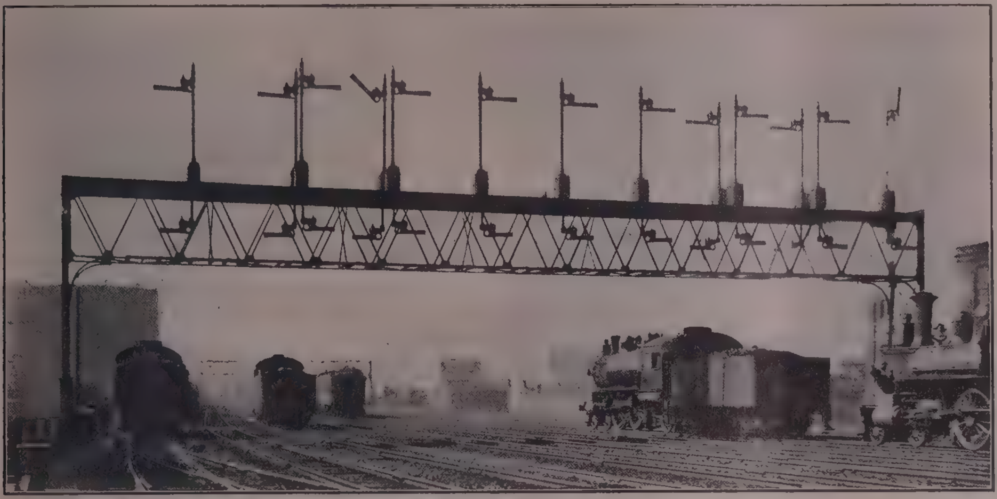
The placing of the mortar or paste, however, does not preclude the necessity of thoroughly cleaning the surface, for if any of the laitance or partially set cement is left on the old concrete, it will form a joint under the new mortar through which water will pass.

A long wall or a conduit may require longitudinal reinforcement to prevent temperature cracks and therefore leakage through them.

The Rainfall in New York City during May amounted to 9.1 in., the greatest for that month during any of the 38 years that the Weather Bureau has maintained a station in the city. On May 7 and 8 there was a rainfall of 4.16 in., the greatest for any 24-hour period during any May since the records began. The greatest rainfall on record for 24 hours in the city was on Oct. 8 and 9, 1903, amounting to 9.4 in.; the next record for the same period was 6.17 in. on Sept. 23, 1882, and the third record was 4.86 in. on Sept. 18 and 19, 1894.

The New Interlocking Signal System at the Broad Street Terminal, Philadelphia.

Extensive changes in the trainshed yard layout have recently been made at the Broad Street terminal of the Pennsylvania R.R. in Philadelphia, for the purpose of accommodating the greatly increased traffic at that point. During the past five years the traffic has increased here about 8 per cent. annually, the total number of passengers handled in both directions in 1907 numbering over 21,000,000. The regular train movement in and out of the station per 24 hours has now increased to 568 trains for each week-day.



View of Signal Bridge No. 4, Showing the New Form of Signal.

As a result of this growth of traffic, it became necessary to provide additional trackage and other facilities at the terminal for increasing the speed at which trains may be handled. Owing, however, to the limitations of the right-of-way entering this terminal, through the densely built-up section of the city, extensions of the station-yard layout were not possible, except for a slight increase in the length of the trainshed tracks and platforms. The latter was provided by the removal of a small building at the outer end of the trainshed on the southerly side, and this, with a revision of the trainshed track switching connections, has permitted a considerable increase in the length of the tracks in the shed; the shortest track in it is now 600 ft. in length, as compared with 480 ft. before the change, while the more important ones have been made 1,000 ft. long, and now 191 cars may be accommodated on the trainshed tracks as against 152 cars before the enlargement. Further than this, no changes in the yard trackage were possible, the yard layout consisting of 16 trainshed tracks approached by 8 tracks on the viaduct leading from West Philadelphia.

Increased speed in the handling of trains was therefore sought by the new system of speed signaling, that has been tried out at other points on the Pennsylvania system and to which reference has previously been made in this journal (March 16, 1907). This system involves the use of three position signals, with signal arms moving in the upper quadrant, and with all of the high signals semi-automatic in action. With the new system of signals, there has also been introduced the use of track circuits for locking the switch levers, enabling the complete elimination of mechanical detector bars on the tracks at switching points in the yard, an arrangement which was introduced in its present form at several electro-pneumatic interlocking plants on the Pennsylvania R.R. in 1903-4, and has been applied at other plants on the system since that time. These novel features combine to render the plant one of the most modern and interesting signaling and interlocking installations in use.

View of the Rearranged Trainshed Yard at the Broad Street Terminal, Philadelphia.

The present layout of the station-yard tracks is shown in the accompanying plan-drawing. The trainshed tracks converge sharply into six tracks on the approach viaduct at Eighteenth Street, approximately 650 ft. west of the outer end of the trainshed, which form the main connections to the trainshed tracks, the seventh track to the south being a siding. Beyond Eighteenth Street there are two ladder tracks, which connect the eight tracks that extend westward from this point, and have double slips at every crossing. The yard layout is greatly simplified by the absence of coach and engine storage tracks, there being, in fact, but three stub-tracks and the siding above referred to at the south side of the trainshed track leads, which are used for anything but passenger traffic, these tracks being assigned to the express company. There are in the portion of the yard devoted to passenger traffic, 34 single switches and 20 double slip switches, which it is necessary to protect for through movements. These comprise that portion of the station yard and approach tracks lying approximately 2,000 ft. to the westward of the trainshed, and they are operated from a signal tower, lying on the northerly side of the yard, about 400 ft. west of the outer end of the trainshed, which was used in connection with the former system and is known as cabin A. It is proposed, however, to relocate the interlocking machine in a new cabin located some 250 ft. to the westward upon the Eighteenth Street bridge, which would be a more central location with respect to the station yard and thus give the signalmen a clearer view of the train movements.

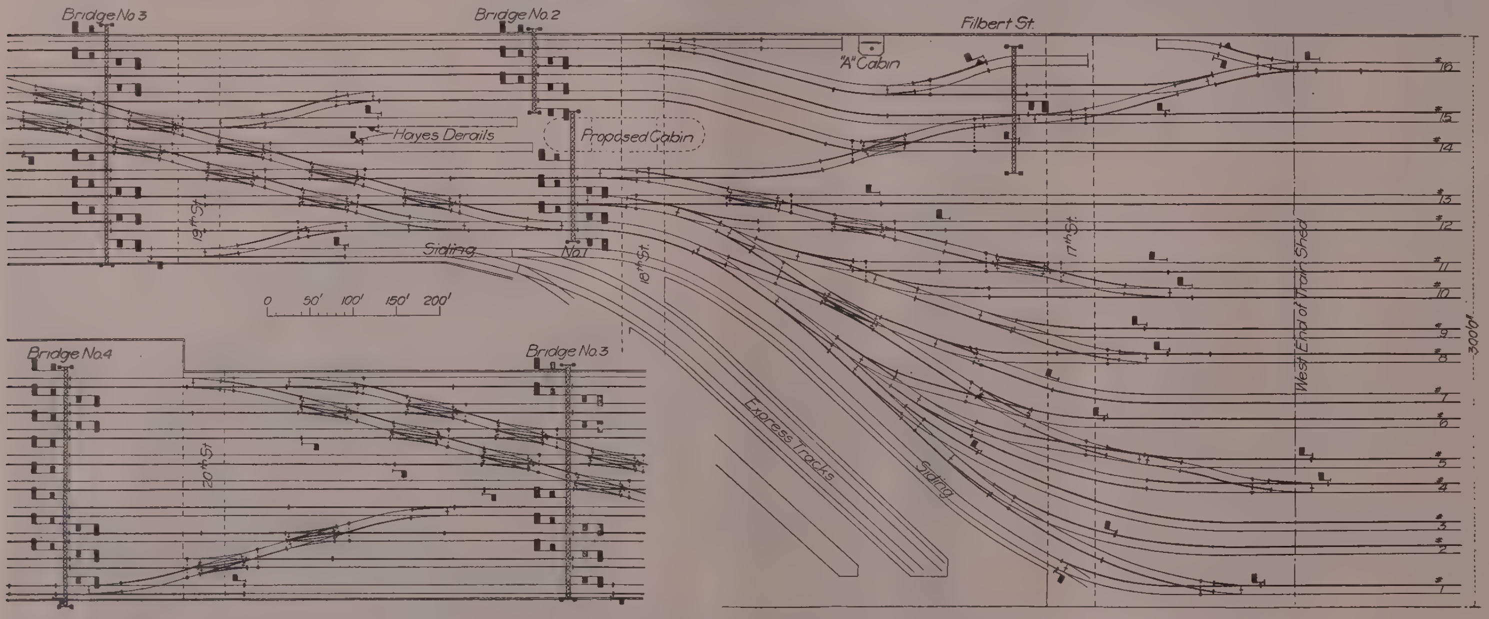
The switching and signalling of the yard is operated by an interlocking equipment on the Westinghouse pneumatic system, which embraces the usual features of electric control of switch and signal movements and mechanical locking, but at the same time has incorporated with it the new feature of route-locking applied by means of track circuits, and on the machine, track circuit indication lamps which are arranged in connection with the switch levers to indicate the passing of trains over the track circuits. The interlocking machine has a total of 107 levers, of which 18 are spare levers, held in reserve, and of the remainder, 47 switch levers operate 158 switches and slips, with movable point frogs, and 42 signal levers control 100 signals. The machine is of the construction that is usual with the Union Switch & Signal Co., for the electro-pneumatic system, with the exception that, owing to the multiplication of circuits introduced by the use of track detector circuits for locking, the contact rollers at the rear of the machine have been placed in vertical position to provide adequately for the large number of contacts necessary on some of the rollers. The rollers are connected with the main horizontal lever shafts through simple bevel gear drives, and with the contact plates in this vertical position at the rear of the machine, the combination has proven very accessible and convenient. The track circuit indication lamps consist of small incandescent lamps arranged for each of the switch levers of the machines which are lighted and extinguished by the action of the track circuits in the yard directly; they so operate as to extinguish all lamps of the switches involved in the route when trains pass the signal of the route, and to light each lamp up again as soon as the train has passed from its track circuit and cleared its switch, the lamps then remaining lighted as long as the switches are clear.

One of the novel features in this plant is the method of route locking used, which consists of the locking through the medium of the track circuits of all switches in a route when trains pass the signals of the route, and the release of each switch in the route by the train as the latter clears the track circuits governing it. Route locking, as heretofore applied, referred to automatically locking the entire route when a train passes the signal and holding the route locked until the entire train has passed completely over the route. But because of holding all switches in the route locked until the last pair of wheels of the train has passed off the entire route, that form of route locking was precluded from use in busy terminals similar to this, and in complicated track layouts, where step by step movements of trains are frequent; where interchange of traffic is the rule rather than the exception in operation; and where shunt or drill movements are very prevalent. Under such conditions switches are frequent and usually lie along the entire route which may be set up, and should the locking be held under the above conditions in the rear of a train passing over the route, switches which might be advantageously shifted in the rear of this train for other train movements that could be made with entire safety, would be still locked up, and hence serious tie-ups in the operation of the plant would result.

The method of route locking as here applied with the releasing feature, whereby switches in the rear of a train on a route are released as fast as the entire train clears the fouling limits, permits of the application of this locking to this class of service. In this system all track circuit relays are repeated through relays in the tower, and these relay contacts intervolved with the spring combination on the machine—the well-known method of controlling circuits in the electro-pneumatic type of machine, constitute the medium of controlling these electric detector switch locking and route locking circuits. The actual locking of the switch levers is accomplished through the medium of the indication magnetic locks on the machine, which lock the switch levers in the normal or reversed position in the usual manner.

Mechanical detector bars have, with the use of the track circuits for automatic switch locking, generally known as "electric detector switch locking circuits," been entirely eliminated from the switches of the yard layout, and this method of locking the switches while trains are passing over them is considered a great improvement over the detector-bar method. In large and complicated track layouts, such as the one here described, the problem of properly applying detector bars to switches, including double slip switches and movable point frogs, becomes somewhat difficult, and their application involves the use of considerable apparatus in the way of moving connections and apparatus in the vicinity of the switches and along the rails adjacent to and forming the switches. With the use of the detector locking circuits, this entire detector-bar apparatus is eliminated—enhancing the appearance of the tracks involved in the interlocking considerably and obviating the troubles from packing with snow and ice in winter.

Signal Bridge No. 3 and the Outer Yard.



Plan of the Broad Street Terminal Yard Showing Location of Signals and Arrangement of Track Circuits.

The other novel feature of the installation is the use of the new form of three-position signals with arms moving in the upper quadrant. It is essentially a form of speed signalling, provisions being made in this system for three different speeds of train movements, thus enabling high-speed, medium speed and low speed, the latter for movements where the speed of the train must be kept absolutely under control, as in cross-over or switching movements. Owing to the proximity of this plant to the terminal, however, only medium and low speed movements of trains were practicable and accordingly, but two signal blades were provided on each mast, the upper arm authorizing medium speed movements, which are limited to 15 miles an hour in the terminal and the lower arm, which is of shorter length, low speed movements under absolute control. The signal arms are located uniformly 12 ft. apart on the mast, the upper arm being mounted on a section of the signal mast projecting above the signal mechanism box which rests on the signal bridge, while the lower arm is mounted on a suspended mast about 3 ft. below the bridge floor. Both arms have the horizontal or stop position as their normal indication, the two other positions consisting of an inclination of 45 deg. and pointing vertically upward, although the lower arm is never raised above the 45-deg. position.

The significance of these indications is, as stated in the previous article upon this system of signalling (March 16, 1907), as follows: Horizontal position of red light at night—stop; arm inclined at 45 deg. or green light at night—proceed with caution to the next signal, at which be prepared to stop; pointing vertically upward or white light at night—proceed at speed on a clear track, the next signal being at clear or caution. As the arms remain normally horizontal in position and showing red light at night, and are in all cases moved one at a time, an engineer cannot pass a signal bridge unless the signal for his particular movement is at caution or clear, the other signal on that post being invariably at danger. The lower arm inclined 45 deg. upward indicates that the engineer may proceed with caution expecting to find the track occupied between that signal and the next. The method of raising the arm above horizontal, for the caution and clear indications, has distinct advantage over the usual practice of lowering it, in that accumulations of ice and sleet during winter storms tend to throw the signal to danger rather than to caution or clear.

An advantage of the use of the track detector circuits and route locking is that all high signals are rendered semi-automatic, every signal arm controlling a route, going to danger immediately on the passage of a train through its track circuit. These signals cannot then be cleared until the train has passed out of the track circuit controlling that arm and in addition the signal lever on the interlocking machine has been put back to normal. Thus all signals are kept in the danger or stop position except as the signalmen clear them and the signals must be cleared for each and every train movement.

The switch and signal mechanisms are of the usual construction of the Union Switch & Signal Co., slight modifications only being necessary in the blade mounting and operating mechanism. Six and eight point relays are extensively used in the plant, there being in all, 434 relays used of both the six and eight point types. The repeater and locking relays, which are operated at 16 volts, are wound for 1,000 ohms, and the track circuit relays are wound for five ohms. There are about 800,000 ft. of No. 12 insulated wire used in the installation.

For the supply of current to "A" tower, that controls the station yard, there are two independent groups of storage batteries, one to supply current for track circuits and the other for the machine, the indicators, block instruments and electric light indications. Both the track circuits and machine are dependent ordinarily upon the batteries, but can, when occasion arises, be supplied direct from the generators. The batteries supplying current to track sections consist of two groups, each group consisting of two sets of six calls in series, the two sets being in multiple, of the 9E type. They are so wired that one group is charging while the other is discharging, there being a constant discharge of 16 amperes. The groups are shifted from charge to discharge every twelve hours. The batteries supplying current for the machine, indicators, indicating lights, etc., consist of two groups of Chloride 9E cells, in series multiple and so wired that one group is charging while the other is discharging. There is a constant discharge of 30 amperes and the groups are shifted every twelve hours.

At present, the batteries for track circuit work are charged from a mercury arc rectifier of 25 volts and 50 amperes capacity, while those for the machine are charged from two alternating-current motor-driven generators of the vertical type, each generator having a capacity of 25 volts and 25 amperes. The installation of the final arrangements for charging these batteries will soon be completed and will consist of two mercury arc rectifiers of 25 volts and 50 amperes capacity, one for charging the track batteries and the other for the machine batteries. There will be also a direct-current 110-volt motor of 5-h.p., with two independent generators coupled to the same shaft, each generator of 25 volts and 50 amperes capacity. One generator will furnish current for track circuit batteries, and the other for the machine batteries, and this outfit is to be run ordinarily while the batteries are being charged and the mercury arc rectifiers will be held in reserve for emergency use.

The new interlocking plant was designed by the signal department of the Pennsylvania R.R. under Mr. Geo. D. Fowle, formerly signal engineer. Mr. A. H. Rudd is now signal engineer, and Mr. C. C. Anthony, who devised the new track detector circuit system of locking, the assistant signal engineer. Mr. W. P. Allen, inspector of signals, had charge of the circuit plans. The equipment was installed under the direction of Mr. J. E. Gillmor, supervisor of signals, Philadelphia Terminal Division, and under the immediate supervision of Mr. B. F. Ohler, assistant supervisor of signals at the Broad Street terminal, Philadelphia.

The Rubble Concrete placed in the Granite Reef dam of the U. S. Reclamation Service during March amounted to 1,977 cu. yd., of which 437 cu. yd. consisted of rock imbedded in the concrete. According to the "Reclamation Record," the total expense connected with the quarrying, hauling and placing of the rock was $2.70 per cubic yard. The itemized cost of the concrete was as follows: Building forms, 30 cents; hauling gravel, 67 cents; hauling cement, 12 cents; concreting, 61 cents; maintenance and operation of track and chute, 5 cents; power, 4 cents; cable expense, 17 cents; imbedded stone, 16 cents; general expenses, 35 cents. This makes the total cost of the concrete, exclusive of cement, amount to $2.47. The cement used in the work cost approximately $1.85 per barrel. Laborers were paid $2 and $2.25 per day, carpenters at the rate of $3 to $4 per day, and teamsters at the rate of $2.25. The item of general expense includes the proper proportions of the general office charges, the salaries of the constructing and assistant engineers and their parties, and the wages of the general foremen and the time-keepers.

The Navigation Resources of American Waterways.

The present condition of the water transportation system of this country and of English, French and Germany waterways was summarized by Prof. Emory R. Johnson, of the University of Pennsylvania, in an address for the recent conference on the conservation of natural resources at the White House. The author's high standing as a specialist in transportation and commercial affairs gives much weight to the following statements abridged from that paper.

The inland waterways of the United States comprise about 25,000 miles of navigated rivers, a nearly equal mileage of streams that can be made navigable by the improvement of their channels and the regulation of the flow of their waters, the five Great Lakes with a combined length of 1,410 miles, and 2,120 miles of operated canals. In addition to these rivers, lakes and canals there are 2,500 miles of sounds, bays and bayous, capable of being converted by means of connecting canals aggregating less than 1,000 miles in length, into an inner route for the coastwise traffic of the Atlantic and Gulf. The waterways in our country—rivers, canals, lakes and coastal channels—have an aggregate length of between 55,000 and 60,000 miles, and only about half of the entire mileage is now used for navigation.

The total appropriations made by Congress from the beginning to 1907 for the rivers of the Mississippi Valley amount to $208,484,720. Over $17,000,000 of this total were expended upon the Mississippi River, between New Orleans and the Gulf, and had reference rather to maritime than to inland commerce; and on some rivers, particularly the Mississippi, appropriations have been largely spent in levee construction and other work which, while indirectly helpful to navigation, was intended primarily to prevent the rivers from destroying the lives and property of those living on or near its banks. With the exception of the Ohio (including the Monongahela and Kanawha) and Mississippi and a few other large rivers, relatively little has been expended since 1890 by the Federal Government in bettering river navigation. The United States has spent during the past hundred years in regulating, improving and extending our system of natural waterways only 4¼ per cent. of the amount private capitalists have invested in the construction of railways.

Portions of 23 streams have been canalized to an aggregate length of 1,520 miles in 1906. This was a gain of but 442 miles over the figures for 1889.

The United States operated 12 canals in 1906 with a combined length of 78.19 miles. These, however, were constructed to overcome obstructions to lake and river navigation and were not independent waterways. In 1907 the Hennepin Canal from the Mississippi River at Rock Island to the Illinois River at its great bend was put in operation by the United States, and this waterway, 96 miles long, is the first of its kind to be managed by the Federal Government. The only really important State canal is the Erie, now being modernized by the people of New York.

Provision was made 21 years ago for the systematic collection and regular publication of detailed statistics of the railroads and their traffic; but as yet Congress has provided no machinery for recording and making public the facts concerning either the commerce moved or the craft operated coastwise or upon our inland waterways. The highly valuable reports of the United States Commissioner of Navigation contain classified information regarding documented American vessels, the aggregate tonnage of which is about six and a half million tons; but there is an equal tonnage of undocumented craft not included in the tables published by the Bureau of Navigation. Comparatively few people are aware of the fact that American shipping has a total tonnage of over 13,000,000 tons gross register. Similarly, the publications of the Bureau of Statistics contain no statistics of the mileage of our inland waterways, no information as to the commerce moved coastwise, very little data regarding river traffic, and only such figures concerning canal shipments as are collected by the States. The statistics of the traffic on the Great Lakes are compiled, but even these statistics are acknowledged not to cover all the commerce moved on the Lakes. The Bureau of the Census has about completed a report on "Transportation by Water" in the United States in 1906. This, like the preceding report of that Bureau made in 1889, is a valuable document, exceptionally complete as regards American shipping, but necessarily less satisfactory regarding passenger and freight traffic, for the simple reason that full and accurate information regarding traffic cannot be obtained until machinery shall have been provided for the systematic and daily recording of freight and passenger movements.

The most rapid growth since 1889 has been in the commerce of the Great Lakes, which rose from 25,266,978 tons of shipments in 1889, to 75,610,690 tons in 1906. The port to port traffic of the Mississippi River and its tributaries in 1906 amounted to 19,531,093 tons of freight. There was also handled locally in and about the harbors, 8,325,548 tons, making a total of 27,856,641 tons for the rivers of the Mississippi Valley. In 1899 the figures were 29,401,409, there having been a decrease of 1,544,768 tons in the 17 years. The freight handled on the other inland waterways of the United States experienced a very large decline during this period, the total tonnage of freight carried having fallen from 11,221,224 tons in 1889 to 3,944,655 tons in 1906.

In order to make the statement of traffic complete the passenger business must be included. The highly efficient steamers of the lake lines carried 14,080,146 passengers in 1906; the figures for 1889 were 2,235,993, the increase during the 17 years being 529.7 per cent. There was also an increase of 30 per cent. in the passenger traffic handled on the Mississippi River and its tributaries, the figure for 1889—10,858,894—having risen by 1906 to 14,122,241. It should be noted, however, that this increase was due more largely to the growth in short distance travel and in ferry traffic than to an increased patronage of the river steamboats operated over the longer routes. The passenger traffic on the other inland waterways of the United States in 1889 is not known, but it amounted to 1,877,889 in 1906.

A comparison of the United States with some of the leading countries of Europe as to the extent to which the navigation resources of each country have been developed, and as to the traffic uses made of inland waterways, will contribute to a clearer appreciation of the facts concerning the United States.

Great Britain.—Although Great Britain is of small territorial area and has not followed the policy adhered to by the continental countries of retaining the inland waterways as public highways to be developed by the government, the mileage and traffic of her streams and canals are by no means insignificant. The tidal coast line of Great Britain, 3,900 miles in length, is supplemented by about 4,000 miles of canals and improved rivers. The traffic on these inland waterways, exclusive of the coastwise maritime commerce, amounted to 37,426,886 tons in 1898, the latest year for which official Board of Trade statistics are obtainable.

The five principal estuaries of England—the Mersey, Humber, Wash, Thames and Severn—are connected by 19 through canal routes. Nine of these through routes reach Severn ports, nine have London termini, ten reach Liverpool and five terminate at Hull. The traffic is most active on the canals in central and northern England. The 642 miles of waterways in this highly developed industrial district moved 23,500,000 tons of freight in 1898. The canals reaching Birmingham had 7,750,000 tons of traffic.

There is a widespread sentiment in the United Kingdom that the country has made a mistake in permitting its waterways to pass largely into the control of the railroads. Somewhat over two years since a Royal Commission on Canals was appointed to make a thorough investigation of the inland waterways of the country and their uses for navigation and other purposes. This body has gone about its work with the thoroughness characteristic of British royal commissions, and a comprehensive report will soon be issued.

France.—France has developed her inland waterways more systematically and completely than has any other of the larger European countries. There are 3,062 miles of canals now in operation and 4,500 miles of rivers—largely as the result of canalization—are used for navigation. Prior to 1880, the greater portion of the freight business of the country was handled on the waterways. Since then the development of the railroads has naturally enabled them to exceed the waterways in tonnage; but the waterways have, none the less, transported a steadily increasing tonnage. During the 20 years from 1885 to 1905 the freight tonnage rose from 19,573,000 to 34,030,000 metric tons. The ton-milage of the water-borne traffic amounted to 3,178,000,000 in 1905 and was more than double the total for 1885. All the waterways of the country, with the exception of 160 miles (75 of which are owned by the city of Paris), are public ways improved and maintained by the State. Their use is free, toll having been abolished in 1880.

Germany.—The policy of Germany in the development of her inland waterways is peculiarly instructive. The canals and navigable rivers of that empire, not including the tidal portions of the rivers navigated by sea-going vessels, have a total length of about 7,600 miles, of which about 6,250 miles may be considered to be commercially important. Three-tenths of the 6,250 miles consist of canals and slack-water river navigation, and seven-tenths of river courses improved without the construction of dams and locks. The railway mileage of the country is 34,000, somewhat over six times the length of the waterways actively used for commerce.

The tonnage of the shipments and receipts of freight transported on the German waterways in 1875 was 20,800,000 metric tons. The figures for 1905 were 103,400,000 metric tons, there having been an increase of nearly 400 per cent. The ton-mileage of this water-borne commerce in 1875 was 1,812,500,000, and in 1905, 9,375,000,000—a gain of over 400 per cent. The ton-mileage of the railroad freight traffic in 1875 was 6,812,500,000, and in 1905 the total was 27,875,000,000. The gain in the railroad traffic was far greater absolutely; but the percentage of increase in water traffic was larger, although the length of the waterways in 1905 was not much greater than in 1875, whereas the railway mileage had more than doubled during that period.

The density of traffic on the German waterways much exceeds that on the railroads. The number of tons carried one mile per mile of waterway rose from 290,000 in 1875 to 1,500,000 in 1905; the corresponding figures for the railways were 410,000 tons carried one mile per mile of line in 1875 and 820,000 in 1905. The relative importance of the traffic of the railways and waterways in Germany is shown by the fact that 25 per cent. of the total ton-mileage of rail and water traffic in 1905 was water-borne, and 75 per cent. moved upon rails. Thirty years earlier the waterways had 21 per cent. and the railroads had 79 per cent. of the combined ton-mileage.

The primary reason for the decline in the use of American canals of such small dimensions and river channels so shallow as to permit the use only of craft capable of transporting 100 to 200 tons of cargo, is to be found in the very success which the railways of the United States have had in providing cheap transportation for heavy and bulky commodities. In no other country of the world have rail transportation costs been reduced to such a low figure. The ability of our railroads to handle this class of traffic has resulted largely from the fact that by far the greater share of the tonnage of American railroads consists of such bulky commodoties as coal, iron ore, lumber and grain, which can be handled not only in carload lots but in train loads and which, from the very size of our country, must be moved long distances in order to reach the manufacturing centers of the United States and the primary markets within and without our borders.

In many other countries it has been found more economical to do the heavier transportation work by making large use of waterways, and to develop the railway traffic more particularly with reference to the speedy movement and schedule delivery of parcels, packages and general commodity freight. This organization of the transportation service by rail is possible where there is a division of the transportation work between the railroads and waterways, and it results in the close co-ordination of railroad freight traffic with the wholesale and retail trade. It enables merchants and manufacturers to reduce capital costs and warehousing expenses to a minimum. It meets the needs of densely populated and highly developed industrial countries, such as France and Germany, and especially of such a country as Great Britain, for, although the inland waterways of Great Britain are, as a whole, less carefully developed than are those of France and Germany, a large share of the domestic commerce of the United Kingdom is carried by water. The navigation services which most countries can secure only by means of inland waterways, the island of Great Britain, with its 3,900 miles of tidal coast, obtains from the surrounding ocean, the best of all highways.

Such an organization of the business of transportation as has been worked out in the three European countries just mentioned, does not result in as low average freight rates by rail as prevail in the United States; but the costs of wholesale and retail distribution and of many manufacturing activities are undoubtedly less than they would be were the people of Europe served almost entirely by railroads and not by railroads and waterways. Our dependence upon railroads, almost exclusively, for the movement of bulky commodities long distances even at low average rates, while we at the same time neglect the development and use of our inland waterways, does not necessarily mean that we have organized our work of production and distribution in the most economical manner. Indeed, there can be little doubt that as social and industrial conditions in the United States approach more closely those prevailing in Europe, we shall find it increasingly desirable to provide ourselves both with well-developed waterways for handling much of our bulky traffic, and with railroads more efficient than present conditions permit them to be in the handling of package freight. We, as well as Europe, will find it profitable to minimize capital and warehousing costs.

The construction of canals and the improvement of rivers in the United States have progressed slowly, in part at least, because the Federal Government has in the past left to the States the work of canal building, and, to some extent, the canalization of rivers. Several of the States, after having made costly mistakes in the execution of their earlier works of internal improvement, have been disposed to leave to private capital the creation of such waterways as special business interests might find it profitable to establish. In many instances the States have thought best to dispose of their waterways to the railroad companies, which have generally found it unprofitable to maintain both rail and water routes. The policy of leaving the development of water transportation to any considerable extent, either to the States or to private corporations, is now realized to be wrong in theory and unsatisfactory in practice.

Tests of a Street-Cleaning Machine in New York City.

A new street cleaning machine for cleaning sheet asphalt is being tested by the Department of Street Cleaning, New York City, in the Borough of Manhattan. It consists of a watering device, behind which is a roller or "squeegee," to which stiff rubber strips are fastened in spiral fashion.

The machine is about 12 ft. long over all and carries a 500-gal. water tank. The three sprinkling pipes are located between the front and rear axles, and the roller which does the sweeping is hung from the rear axles on a frame behind the machine. The roller consists of a core, 10 in. in diameter, into which the rubber strips are fastened. There are twenty-four of these strips, each ⅜ in. thick, projecting 4½ in. from the core, so that the size of the roller is about 19 in. The strips are placed spirally and this, combined with the stiffness of the rubber, keeps them from turning down easily or "flopping." The roller is 7 ft. long and placed at an angle similar to the revolving brooms of the ordinary street sweeper. It is revolved in a direction opposite to the movement of the machine by gearing from one of the rear wheels.

The trials in New York City began on June 5, and have been confined to Fifth Ave. and the cross-town streets south of 44th St., from First Ave. west to Sixth Ave. The number of yards swept per day during the first four days and the number of tanks of water used are shown in the table.

	Square Yards.	Tanks.
June 5	50,000	12
June 6	40,559	10
June 8	46,888	11
June 9	54,833	11

These figures show an average consumption varying from 1-10 to 1-8 of a gallon per square yard. The work on June 5 and 9 represents about 7 hr. of actual work on the pavement, and on June 6 and 8 about 6 hr.

While the department has not as yet drawn any conclusions as to the operation of the machine, the results in general seem to be satisfactory. The fine dust, horse droppings, oil, and slime are removed better than by hand sweeping or revolving-broom machines. The rubber strips appear to bear with considerable force against the pavements and to enter the small hollows and waves of the surface. The amount of pressure is indicated by the fact that the strips pick up a great number of small pins, which are deeply imbedded in the rubber.

The machine is built by the Kindling Machinery Co., Milwaukee, Wis.

A 4,000-h.-p. Steam Engine of the double horizontal type at the central electric generating station at Brussels, Belgium, has shown a consumption of 9.6 lb. of steam per indicated horse-power hour with loadings from half to full load. The engine, which was built by the Van Den Kerchive Engine Co., of Ghent, has valves of the piston type, vertically arranged and operated from a lay shaft, and has 34½ and 60 by 60-in. cylinders. The ratio of areas of high-pressure to low-pressure cylinders is 1 to 2.97, the speed, 83 r. p. m. and the piston speed, 828 ft. per min.

Experiments on the Preliminary Treatment of Potomac River Water at Washington.

Abstract of a Report to Major Spencer Cosby, Corps of Engineers, U. S. A., by Francis F. Longley, Chief Chemist and Assistant Superintendent, Washington Filtration Plant.

When the question of the purification of the Potomac water for the District of Columbia was before Congress, controversy arose as to the best means of purification. The final note of the controversy was sounded in the report of Messrs. Hering, Fuller and Hazen, of Feb. 18, 1901, in which they concluded as follows:

"In consideration of the full evidence, we recommend the construction of a complete system of slow, or sand filters, with such auxiliary works as may be necessary for preliminary sedimentation and the use of a coagulant for part of the time."

Subsequent to this report, appropriations were made for the construction of slow-sand filters, provision being omitted for the preliminary works upon the recommendation of the late Col. A. M. Miller, dater Feb. 28, 1903, based upon the understanding that they could be installed, if desired, after a period of operation of the slow-sand filters.

Sedimentation Basins and Building at Experimental Plant.

A year's operation of the filters showed results that were generally satisfactory, but with room for improvement; and in view of this and of the conditions above mentioned, it was decided to institute these experiments as a basis for further recommendations if called for.

Operating Results at Main Plant.—The immediate excuse for the construction of the Washington filters was the high typhoid rate. Therefore, the point hoped for primarily in the operation of the plant was a bacterial content beyond criticism.

The average bacterial content of the filtered water has been, for months at a time, from 10 to 20 bacteria per cubic centimeter, thus proving the capability of the filters for an excellent purification under proper conditions. The number of bacteria in the filtered water reservoir has exceeded the limit set by the German authorities, of 100 per cubic centimeter, during 7 per cent. of the time, and has at times exceeded 500 or 600; and occasional counts from many of the individual filters have been much higher than that.

The bacterial condition of the water from the filtered-water reservoir, as determined by gelatine counts, may be seen from the accompanying table.

The occasions of high bacterial content are coincident with periods of high turbidity in the raw water; and from the results obtained at the experimental filters, given in one of the accompanying tables, it is clear that these periods of high bacterial content in the filtered water might be entirely avoided by proper preliminary treatment.

Table 1.—Daily Bacterial Content and Turbidity in Filtered-Water Reservoir at Main Plant.

Bacteria. Nov. 1, 1905 to Feb. 1, 1908. Count between		No. of Days.	Turbidity. Oct. 7, 1905, to Jan. 1, 1908. Turbidity.	No. of Days.
0-20	per cubic centimeter	291	1 or less	438
20-40	" " "	245	2	116
40-60	" " "	63	3	40
60-80	" " "	30	4	29
80-100	" " "	28	5	46
100-200	" " "	29	6	47
200-300	" " "	13	7	23
300-500	" " "	5	8	23
500-1000	" " "	5	9	7
			10	14
			11-12	17
			13-15	11
			16-20	5

To the majority of consumers, the removal of the large quantities of clay that used to fill the system at frequent intervals has been more heartily appreciated than the removal of bacteria. A thorough removal of this clay, therefore, was also desired. When the turbidity of the water applied to the filters is low, and conditions are favorable for a good removal, the turbidity of the filtered water is almost zero. During times of high turbidity in the applied water, however, some of the clay goes through into the effluent and produces decided turbidities therein. During the period of 27 months, from Oct. 6, 1905, to Jan. 1, 1908, the turbidity of the filtered water exceeded 5 for 18 per cent. of the time. A turbidity of 5 is quite appreciable in a glass of water and plainly evident in a white bowl or tub; and the higher turbidities, which have been present up to a maximum of 20, are decidedly objectionable.

Object of Experiments.—In the controversy above referred to, the principal method of preliminary treatment considered was occasional coagulation by means of aluminum sulphate, followed by an ample period for chemical reaction and thorough sedimentation before the water reached the filters. It was known, definitely and certainly, that this method would give the results desired. In view of opposition to the use of chemicals in the purification process, however, it was desired to know whether equally satisfactory results could be produced by any means which did not involve the use of a chemical. [A short description of the Experimental Plant and the results of the early months of operation were given in The Engineering Record of May 25, 1907, and a summary of results in the issue of Aug. 3, 1907.]

The essential points of the problem, then, may be stated as follows: 1. To examine into and test devices that involve purely mechanical principles, to determine whether any of these devices are capable of preparing the Potomac water so that when applied to the sand filters, the final effluent obtained will be equal in quality to that obtained by the proposed method of coagulation.

2. To show the improvement in quality that might be expected in the final effluent at the filtration plant, due to the application of the best of the preliminary processes tried.

Experimental Plant.—At the experimental filter plant at Dalecarlia Reservoir there are three sand filters, of reinforced concrete, each one a round tank 10 ft. in diameter and 8½ ft. deep. To these are applied the effluents from three devices for giving the water a preliminary treatment. The construction of the filtering medium of the three sand filters is in all respects similar to that of this plant. The effluent pipes are fitted with meters and are so arranged that any filter may, after cleaning, be filled with water from below, from either of the other filters.

The preliminary device which, with its proper sand filter, is designated throughout this work as "Arrangement No. 1," is entirely similar in construction to a rapid sand, or so-called mechanical filter. It is a circular tank of reinforced concrete, 4 ft. in diameter and 8½ ft. deep. The filtering medium consists of 3 ft. of sand supported on 6 in. of gravel. Beneath this is the brass underdrainage system leading to the effluent pipe. The preliminary filter is placed above the sand filter. Filtration is downward, and the effluent discharges directly into the sand filter. The piping is so arranged that the flow of water may be reversed in order to clean the filter.

The preliminary device which, with its sand filter, is designated as "Arrangement No. 2," was designed in accordance with the ideas of Mr. P. J. A. Maignen. The filtering medium, which is contained in a reinforced-concrete tank similar to No. 1 preliminary filter, consists of layers as follows, from bottom to top: 12 in. of cobble stones; 12 in. of coke, egg size; 12 in. of coke, stove size; 24 in. of coke, nut size, in which were laid four tiers of roofing slate at an angle of about 45 deg. to the vertical; and 24 in. of sponge clippings, compressed under a pressure of 12 to 15 lb. per square foot, and held in place by a wedged grating. This preliminary filter, like No. 1, is placed above its sand filter. Filtration is upward, and the effluent flows directly into the sand filter.

The preliminary treatment used in "Arrangement No. 3" is coagulation. The equipment for this treatment consists, first, of small tanks properly arranged and connected with piping for the preparation and storage of the solution of aluminum sulphate; second, a device for delivering a measured stream of this solution into a measured stream of the water to be treated, and third, two coagulating basins. The water after passing through the coagulating basins flows directly into sand filter No. 3.

The problem of the accurate application of the very small quantities of coagulant solution required for the small amount of water here treated, presented some difficulties, which were solved, as shown in the accompanying sketch. The barrel *B* is perfectly air tight; in the side of the pipe at the desired level, within the "orifice box" *C*, there is bored a ¼-in. hole. As the solution is discharged at the orifice, small bubbles of air enter the pipe through this hole, while permitting the solution to discharge freely into the orifice box through the open end of the pipe. This produces a constant level in the box. The orifices are simply small nozzles made of glass tubing. They were carefully drawn and calibrated in the laboratory to deliver a given volume under a given head in a given number of seconds. A series of these nozzles was made, and the range made continuous by providing for varying the head by sliding the glass tube up or down in the rubber stopper in the bottom of the orifice box. The success of the device depends largely upon keeping it air-tight throughout.

Arrangement No. 3, as first installed, included only a single coagulating basin, 10 ft. wide, 6½ ft. deep and 29 ft. long, with seven wooden baffles the full depth of the basin, placed crosswise in such manner as to leave openings 2 ft. wide at opposite ends of successive baffles. It was expected that this arrangement would give a sufficient period for coagulation and sedimentation. In a general way, 24 hr. was the length of time desired. The first measurement of this period, by means of aniline dye introduced at the inlet, showed that some of the water passed entirely through the basin in 2½ hr. The function of the basin, under such a condition, was, of course, entirely unfulfilled. Experiments with the baffles, and especially the placing of a small baffle near the inlet in such a manner as to form an inlet chamber, designed to prevent the incoming water from mixing with the partially sedimented water, made it possible to hold the water in the basin for 10 hr.

With this one basin it seemed impossible to obtain a period of sedimentation of 24 hr., and therefore another basin, 10 x 6 x 44 ft. was built alongside the first one, and so connected that water would pass successively through both. The period of sedimentation through this new arrangement of two basins was found to be about 45 hr.

The sand used in all three sand filters, and in preliminary filter No. 1, is identical with that used in the filters at the main Washington plant. All the filters are provided with loss of head gauges. The entire equipment, with the exception of the coagulating basins, is enclosed in a neat, substantial wooden building, and proper arrangements provided for heating, lighting and drainage.

For logical reasons, the location assumed for a plant for the preliminary treatment of the Potomac water was between Dalecarlia and Georgetown reservoirs. The experimental plant was therefore located conveniently near the outlet of the former reservoir, where the water supply could be taken directly out of the main stream of flow through the aqueduct at that point.

At this point the water has undergone one day's sedimentation in its passage through Dalecarlia Reservoir. If a preliminary treatment were applied at this point, there would be a subsequent period of sedimentation before reaching the filters, of three to four days more, in its passage through the Georgetown and Washington City reservoirs.

Arrangements No. 1 and 2 were started on Feb. 8 and No. 3 on Feb. 12, 1907. With the exception of necessary stops for cleaning, and occasional alterations or repairs, they have been running continuously down to the present time, and are still in full operation.

Samples and Observations.—For a determination of the bactoriological efficiency of the experimental filters, the necessary samples for bacterial examination have been taken each morning, except Sunday, and carried by messenger directly to the laboratories of the filtration plant. In general, one sample a day has been taken of the raw water, of the effluent of each preliminary filter and of the coagulating basin, and of the effluent of each sand filter.

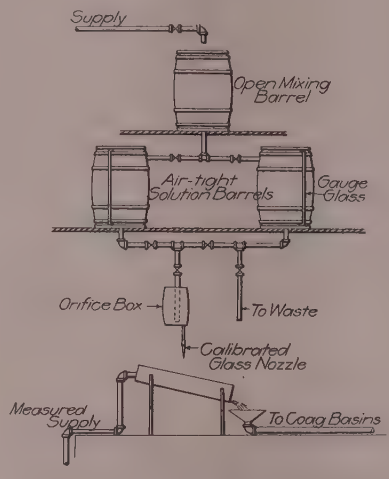
The physical conditions of the various samples of water, for a determination of the clay removal of the different processes, has been determined at the experimental plant. Four samples a day have been taken for the turbidity examination, at the points designated above, and comparisons made, by the operator in charge, with turbidity standards made up and standardized in the laboratories of the filtration plant.

Observations of the quantities of water flowing from the different filters, of the losses of head and of the quantities of coagulant applied, have ben made at intervals frequent enough for the accurate control of all the different parts of the processes. Other special examinations of water, or of the conditions of operation have been made from time to time, and they will be found in the parts of this report to which they pertain.

An examination of records of preliminary filters in use at other places showed a range of rates of filtration, the minimum of which was roughly 50 millions of gallons per acre per day. That rate was therefore adopted for the two preliminary filters herein described.

At the time the experimental filters were started, it had been demonstrated at this plant as well as elsewhere that slow sand filters operated at a rate of 4 million gallons per acre per day showed efficiencies substantially the same as at 3 million gallons.

The three sand filters at the experimental plant were accordingly started at a 4-million gallon rate. Later on, when difficulties were encountered in obtaining a sufficient period of flow in the coagulating basin, the details of which appear earlier in this report, the rate of No. 3 sand filter was reduced to 3 million gallons per acre per day, and it has since been maintained at that rate.



Coagulating Solution Apparatus.

Periods of Service.—The periods of service of No. 1 preliminary filter were relatively long for such a device at first; but the accumulation of clay in the depths of the filtering material and in the underdrains, from frequent washing with muddy raw water, which was the only water available with the requisite pressure, gradually decreased the period of service until the filter would run only one day. Finally all the filtering material was removed and cleaned and the underdrains flushed out, which restored the capacity. Since this cleaning, the arrangement has been put out of service occasionally when it needed washing at times of very high turbidity, to avoid clogging the underdrains again.

The use of this rapid pre-filter No. 1 produced a remarkably long period of service in sand filter No. 1, which would be a strong argument in its favor if there were any other decided advantages in the process.

In the case of preliminary filters of the type of No. 2, the termination of the run is generally determined by the appearance of "springs," or some other equally obvious cause of a decrease in efficiency. After the second run, however, at Mr. Maignen's suggestion, the filter was flushed out at more frequent intervals, to keep the sponge at all times reasonably free of accumulated clay.

The periods of service of the No. 2 sand filter did not differ in any marked degree from the periods obtained at the main plant.

In connection with the question of coagulation as a preliminary treatment, it has been suggested that the elimination of high turbidities might reduce the expense of operation at the Washington filters, either by prolonging the runs or by reducing the depth of penetration and consequently the amount of sand handled.

If the conditions of sedimentation in the coagulating basins had been the same as those in the reservoirs between Dalecarlia and the intake of the filtration plant, the answer to this suggestion should have been indicated with reasonable accuracy. During the first few months of the experiments, however, as has been stated, the sedimentation capacity was very insufficient, and since the new basin was put in service, while this capacity has been increased very materially, it is still less than it should be to make the comparison satisfactory.

Table 2.—Summary of Results of Operation of Experimental Filters for Period of 11 Months.

Arrangement No. 1.	
Preliminary filter—(Rapid prefilter).	
Nominal rate, mil. gal. per acre per day	50
Number of runs	73
Days of service—Max	13
“ “ “ —Min	1
“ “ “ —Av	4
Initial loss of head at 50 mil. gal., in ft	0.8
Total available loss of head	8.0
Av. quantity wash water, gal	2800
“ “ “ “ per cent. of quantity filtered	4.9
Rate of application of wash water, gal per sq. ft. per min	7½
Sand filter.	
Nominal rate, mil. gal. per acre per day	4
Number of runs (2d run unfinished to date)	1
Days of service	228
Depth of sand removed, in	1.44
Arrangement No. 2.	
Preliminary filter—(Maignen Scrubber).	
Nominal rate, mil. gals. per acre per day	50
Number of runs	5
Days of service—Max	91
“ “ “ —Min	20
“ “ “ —Av	55
Initial loss of head at 50 mil. gal., in ft	0.1 or less
Final loss of head—Max	1.22
“ “ “ “ —Min	0.32
“ “ “ “ —Av	0.73
Av. quantity wash water, gal	3500 or less
“ “ “ “ “ less than 0.5% of quantity filtered.	
Sand Filter.	
Nominal rate, mil. gals. per acre per day	4
Number of runs	5
Days of service—Max	85
Daye of service—Min	44
Days of service—Av	61
Depth of sand removed—Max	1.36
Depth of sand removed—Min	.38
Depth of sand removed—Av	.90
Arrangement No. 3.	
Sand Filter.	
Nominal rate, mil. gals. per acre per day	4 3
Number of runs	5
Days of service—Max	93
Days of service—Min	37
Days of service—Av	64
Depth of sand removed—Max	1.88
Depth of sand removed—Min	1.03
Depth of sand removed—Av	1.41

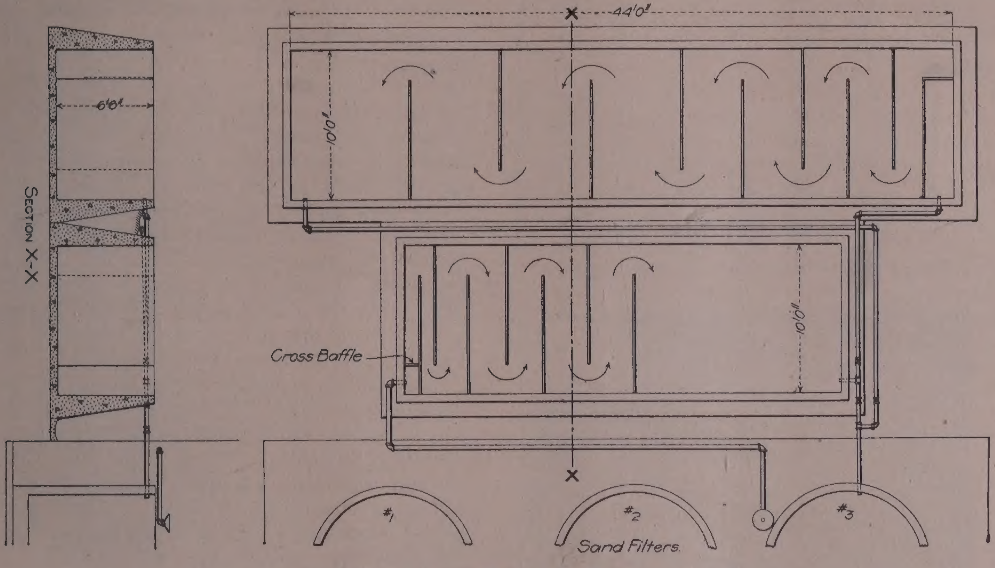
During the early part of the experiments, the runs of No. 3 sand filter were short, because of the suspended aluminum hydrate that was carried to the surface of the sand. During the latter part, the lengths of the runs and the depth of the sand removed, are not consistently different from the figures for the main plant, and therefore there is nothing in these data to indicate any change in expense of operation of the filters following the application of a coagulant.

Additional Experiments.—The preliminary filter in Arrangement No. 2, as mentioned above, was built in accordance with the ideas of Mr. P. J. A. Maignen. He has throughout the experiments shown an interest, and since the results of operation showed that his device would not effect the desired improvement, has offered a number of suggestions aimed at improving these results.

On April 8, finely-ground bone charcoal was applied to sand filter No. 2 at the rate of about 5 tons to the acre. That it was ineffective is shown by the turbidities of the effluents of the sand filters through the next period of bad water. No. 2 filter reached a maximum turbidity of 4, while No. 3 maintained its low turbidity of zero and 1.

On June 25, No. 2 sand filter was again started, with a membrane of asbestos fibre and bone charcoal on its surface. It was later found that the asbestos did not cover the entire surface, so on Sept. 3, the conditions having been remedied, it was started again. As before, the effluent of No. 2 reached a maximum of 4 at the next rise of turbidity in the raw water, while the effluent of No. 3 remained at zero.

Turbidity Results.—The turbidity removed by the different processes and parts of processes is shown in Table 3.



Arrangement of Coagulating Basins of Experimental Plant.

As the monthly averages show, the removal of turbidity by the preliminary process alone was greatest for the No. 3 arrangement, with coagulation at times of high turbidity. During months when the turbidity of the applied water was high, this difference in favor of the coagulating process was very marked.

TABLE 3.—TURBIDITY RESULTS OF EXPERIMENTAL FILTERS.

	No. 1.			No. 2.			No. 3.		
Date.	Raw.	Eff. Pre-lim. Filt.	Eff. Sand Filter.	Raw.	Eff. Pre-lim. Filt.	Eff. Sand Filter.	Raw.	Eff. Coag. Basin.	Eff. Sand Filter.
1907.									
March	81	25	5	81	34	5	81	20	1
April	35	7	2	37	10	2	35	16	1
May	17	2	1	17	3	1	17	9	0
June	45	9	1	43	13	1	43	21	1
July	60	17	1	60	24	1	60	22	0
Aug.	34	6	1	21	4	1	34	15	0
Sept.	88	18	1	109	33	1	116	3	0
Oct.	58	16	1	36	12	1	36	3	0
Nov.	67	18	1	88	31	1	88	8	0
Dec.	50	13	1	132	57	3	142	5	0
1908.									
Jan.	82	34	5	79	40	5	83	3	0
Avg.	56	15	1.8	64	24	2.0	67	12	0.3
% Reduc.		73.2	96.8		62.5	96.9		82.0	99.6

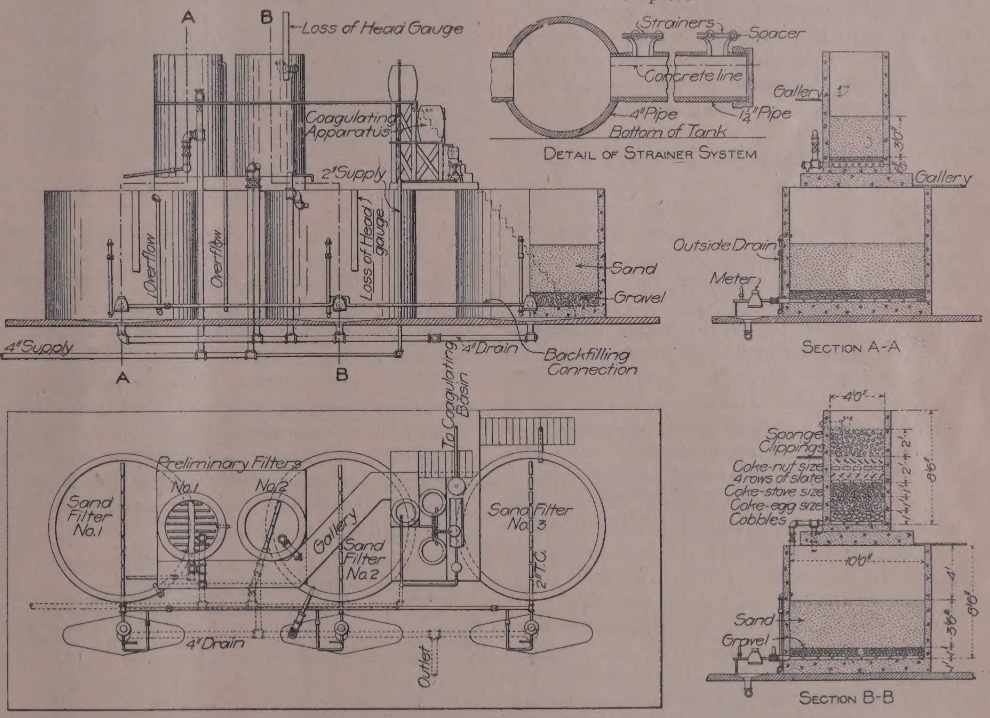
The total removal, by preliminary process and subsequent sand filtration, was also very much more complete for the No. 3 arrangement than for either of the others. This may be shown more clearly by an analysis of the turbidities of the final effluents, and retabulation, as shown in Table 4:

TABLE 4.—NUMBER OF DAYS WHEN STATED TURBIDITIES WERE FOUND IN EFFLUENTS.

Turbidities.	0	1	2	3	4	5	6	7	8	9
No. 1....	2	195	27	12	14	7	8	4	5	0
No. 2....	4	210	24	13	11	8	8	3	2	4
No. 3....	215	81	0	0	0	0	0	0	0	0

Turbidities.	10	11-12	13-15	16-20	20-30	30-40	40-50
No. 1....	0	5	0	2	1	1	0
No. 2....	1	1	1	0	1	0	1
No. 3....	0	0	0	0	0	0	0

This shows, for No. 1 and No. 2 arrangements, a reasonably good removal during the greater part of the time, when conditions are favorable; but high and unsatisfactory turbidities at the very times when the preliminary process should be of the greatest value. A preliminary process is designed for use only at intervals, when conditions are most adverse. If such a process falls down at that time, it is quite useless for the purpose now under consideration.



Experimental Preliminary and Slow-Sand Filters, Washington, D. C.

The figures given for No. 3 arrangement show a turbidity removal satisfactory in every respect. The turbidity of the effluent is not zero all the time, because that is not the primary object in the investigation of this process, and coagulant is not applied all the time. Generally speaking, it is applied only at such times as will effect the elimination of the high turbidities indicated in the effluents of No. 1 and No. 2. This is discussed in more detail under the subject of coagulant.

The most striking comparison between the results obtained from the three arrangements may be seen from an examination of the records for the period of high turbidity during the month of January, 1908. The turbidity results are presented in Table 5.

This shows arrangement No. 1 and No. 2 entirely unsatisfactory for the removal of turbidity during this time while arrangement No. 3 gives a perfect removal. Other periods of high turbidity show results quite similar, differing only in degree.

TABLE 5.—TURBIDITY RESULTS—EXPERIMENTAL FILTERS, DURING PERIOD OF HIGH TURBIDITY, JAN., 1908.

		No. 1.		No. 2.		No. 3.	
	Raw water.	Effl. prelim. filter.	Effl. sand filter.	Effl. prelim. filter.	Effl. sand filter.	Effl. coag. basin.	Effl. sand filter.
Jan. 12.	40	10	1	12	1	2	0
Jan. 13.	110	45	2	51	2	2	0
Jan. 14.	210	95	3	113	4	2	0
Jan. 15.	325	190	12	222	15	3	0
Jan. 16.	360	210	37	247	42	5	0
Jan. 17.	242	122	24	147	26	6	0
Jan. 18.	137	..	..	73	7	6	0
Jan. 19.	117	40	12	*	*	5	0
Jan. 20.	72	31	6	*	*	*	0
Jan. 21.	55	20	4	25	4	*	*
Jan. 22.	49	17	3	21	4	*	*
Jan. 23.	40	12	3	15	3	3	0
Jan. 24.	40	11	3	13	3	3	0

*Cleaning sand filter.

In connection with the question of turbidity removal, the results of some experiments made in double and triple filtration present some interesting data. Two small filters, built on the lines of our large sand filters, were arranged; the first, *A*, to receive water from the filtered city supply; the second, *B*, to receive the effluent of the first. The effluent of *A* was thus twice filtered and that of *B* three times filtered. The rates used were 3 million gallons per acre per day.

TABLE 6.—TURBIDITIES WITH DOUBLE AND TRIPLE FILTRATION.

	Applied water filtered once.	Effluent A filtered twice.	Effluent B filtered three times.
Mar. 1907	6.0	7.0	6.0
April 1907	1.9	1.5	1.4
May 1907	1.1	0.7	0.7
June 1907	0.9	0.3	—
July 1907	1.0	0.2	—
Aug. 1907	1.0	0.5	0.2
Sept. 1907	0.7	0.4	0.2
Oct. 1907	1.3	0.5	0.5
Nov. 1907	1.1	0.2	0.2
Dec. 1907	4.2	2.2	—
Jan. 1908	6.0	3.0	—
Feb. 1908	3.7	1.7	—

Some reduction in turbidity takes place in the second and third filtration, but the removal is by no means complete.

From the evidence at hand, therefore, we may conclude that there will not be at all times a complete removal of turbidity by any of the following treatments:

1. By one day of sedimentation, followed by filtration through a rapid sand pre-filter and then through a slow-sand filter.

2. By one day of sedimentation, followed by filtration through a preliminary filter of the "Maignen" type, and then through a slow-sand filter.

3. By three or four days of sedimentation, followed by filtration through slow-sand filters.

4. By the process described in (3), followed by another and similar filtration through slow-sand filters.

5. By the process described in (4), followed by still another and similar filtration through slow-sand filters.

In other words, from (5), if we consider the entire reservoir system and filtration plant, and even supplemented by another equal area of filters of the same type, as a process for the preliminary treatment of the water only, the subsequent passage of that water through slow-sand filters of that type, will not effect the complete removal of the fine turbidity.

Bacteriological Results.—Table 7 gives a summary of results of bacteriological examination of the water at critical points of the different processes:

Table 7.—Bacteriological Results of Experimental Filters.

Date	No. 1			No. 2			No. 3		
1907	Raw	Effl. Prelim. Filt.	Effl. Sand Filt.	Raw	Effl. Prelim. Filt.	Effl. Sand Filt.	Raw	Effl. Coag. Basin.	Effl. Sand Filt.
March	8295	1852	293	8295	2054	291	8652	2628	118
April	2125	196	30	2197	387	33	2125	1203	28
May	339	21	29	339	68	16	341	756	25
June	978	58	44	985	322	45	978	1587	56
July	580	69	21	580	185	20	580	293	29
Aug.	710	94	12	125	170	11	710	280	15
Sept.	6100	110	13	6200	360	69	6800	425	39
Oct.	2600	85	16	1400	210	27	1400	650	33
Nov.	5500	1100	225	9800	3300	130	9800	1600	90
Dec.	4700	1600	230	20000	5800	550	21000	1200	120
1908.									
Jan.	8750	2081	255	8950	3521	248	9519	728	31
Avg.	3700	660	106	5350	1490	130	5630	1030	53
% Reduc.		82.2	97.1		72.1	97.6		81.7	99.1

The occurrence of Bacillus Coli in the effluents of the three sand filters during four months, October, 1907, to January, 1908, inclusive, was as follows: No. 1, 58 samples taken, 25 positive in 10 c.c., 8 positive in 1 c.c.; No. 2, 85 samples taken, 22 positive in 10 c.c., 7 positive in 1 c.c.; No. 3, 83 samples taken, 9 positive in 10 c.c., 1 positive in 1 c.c.

With the bacteria, as with the turbidity, the total removal was greatest in the case of arrangement No. 3. A classification of bacterial counts was made, to correspond to the turbidity table shown, with results as shown in Table 8.

Table 8.—Number of Days When Stated Bacterial Counts Were Found in Effluents.

Bacteria Arrangement.	0-19	20-39	40-59	60-79	80-99	100-199	200-299	300-499	500-999	1000-1999	2000
No. 1	83	47	31	12	7	14	5	8	6	4	1
No. 2	69	53	29	15	7	23	13	7	5	7	0
No. 3	87	61	13	20	17	33	4	2	0	0	0

This table shows the same tendency as the corresponding turbidity table, but not as plainly, for the reasons given herewith: Storage of water in a reservoir, under favorable conditions, should in general give a reduction of the numbers of bacteria. This statement should hold true of the coagulating basins at the experimental filters if there were no conflicting factors. It has frequently been observed, during periods when no coagulant was being applied, that the numbers of bacteria at the outlet of the coagulating basin were greater than in the raw water.

This may be seen in the table of monthly averages given above. During May, no coagulant was applied. During June it was applied for about two days. The average numbers of bacteria in the effluent of the coagulating basin for these two months were higher than the raw water, while every other month they were lower. As this seemed abnormal, the point was studied in some detail in the laboratory. The results obtained indicated that the increase in bacteria was due to the presence of the wooden baffles in the coagulating basin, with which the water was in contact for many hours.

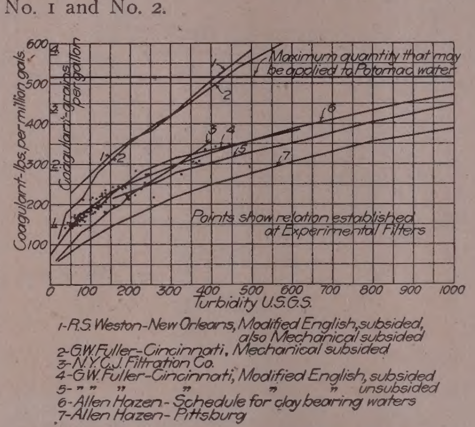
As this condition was evident only during periods of good water when no coagulant was being applied, and as the daily records of bacteria throughout the complete aqueduct system show that nothing of this kind occurs in the storage reservoirs, it may be stated definitely that this small abnormality has no significant bearing upon the problem under consideration, although it obscures the clearness of a general summary.

As in the case of the turbidity, the periods to choose to show to best advantage the bacterial improvement that may result from an application of any preliminary process are the periods of high turbidity and bacteria when such a process would be used, if available. Daily bacterial counts for two of these typical periods are given in Table 9:

Table 9.—Bacterial Results—Experimental Filters, during Periods of High Bacteria.

1907.	Raw.	No. 1		No. 2		No. 3	
		Effl. Prelim. Filter.	Effl. Sand Filter.	Effl. Prelim. Filter.	Effl. Sand Filter.	Effl. Coag. Basin.	Effl. Sand Filter.
Nov. 5	6,000	.....	....	3,000	220	1,600	90
6	5,000	1,500	240	1,100	150	3,900	6
7	14,000	1,000	220	1,600	120	300	9
8	1,900	270	160	2,000	29	230	2
Jan. 6	600	200	37	600	49	230	18
7	1,100	150	47	330	49	370	12
8	1,900	160	30	900	43	1,100	20
9	13,000	1,300	70	3,400	50	1,200	22
10	10,000	3,500	170	8,000	50	700	16
11	16,000	4,000	240	220	200	1,200	11
12	Sunday.						
13	8,500	1,200	340	1,200	43	90	6
14	16,000	3,900	500	6,000	280	150	23
15	24,000	7,000	550	9,500	700	1,100	19
16	28,000	8,500	1,200	14,000	900	1,000	14
17	65,000	15,000	1,700	20,000	1,200	490	23
18	7,000	....	....	6,500	400	1,600	14

These results show the decided advantage of Arrangement No. 3 in the removal of large numbers of bacteria from the water, and the very unsatisfactory bacterial results from Arrangement No. 1 and No. 2.



Quantity of Coagulant for Different Waters.

A comparison of the bacterial removal effected by the preliminary process of Arrangement No. 3, with the bacterial purification in the reservoirs between Dalecarlia, the proposed point of application, and the filtration plant, is of interest in this connection. The following Table 10 will show this very clearly. In order to make the counts represent as nearly as possible the same water, the figures given for the outlet of the coagulating basin are in general one day later, and those for the applied water at the filtration plant three days later, than the counts for the outlet of Dalecarlia Reservoir.

Table 10.—Bacterial Reductions in the Coagulating Basin and in the Reservoir.

	Dalecarlia Outlet.	Coag. Basin Outlet 1 day later.	Applied Water Wash. Filt. Plant 3 days later.
Nov. 22	10,000	2,400	3,800
23	18,000	2,100	3,000
24	....	....	....
25	50,000	2,400	3,400
26	40,000	2,600	3,700
27	16,000	2,500	3,800
28	....	....	....
29	10,000	1,900	2,400
Average	24,000	2,320	3,350
% Reduction		90.3	86.0
1908.			
Jan. 9	13,000	700	2,100
10	10,000	1,200	2,200
11	16,000	90	3,900
12	....	....	....
13	8,500	150	8,000
14	16,000	1,100	10,000
15	24,000	1,000	12,000
16	28,000	490	9,000
17	65,000	1,600	8,500
Average	22,560	790	6,960
% Reduction		96.5	69.1

The conditions following coagulation are characterized by a generally sharp and considerable reduction of bacteria, in comparison with the reduction that takes place from natural causes in the reservoir system.

It may be concluded, therefore, from the data presented herewith, that the mechanical devices under investigation are incapable of giving at all times the bacterial removal desired of such a preliminary process; and from these data, supplemented by the records of operation of a number of thoroughly successful purification plants in which coagulation is an essential factor, that this process applied to the preliminary treatment of Potomac water will give a bacterial removal thoroughly satisfactory in all respects.

Coagulant Used.—The coagulant used in these experiments was aluminum sulphate. For the purpose of properly estimating the quantities of coagulant that would be required if this preliminary process were to be applied on a large scale, it is necessary to arrive at some limiting turbidity, below which it will not be necessary to coagulate the water. It was expected that this limiting turbidity could be determined, for practical application, by the results from the coagulating arrangement at the experimental plant. The time, however, required for water to pass through the two reservoirs from Dalecarlia to the intake of the filtration plant is about three days, while the longest period that was obtained in the new arrangement of basins was 45 hr., or less than two-thirds the desired period.

The limiting turbidity as determined at the experimental filters, therefore, is not a satisfactory limit upon which to base estimates for actual operating conditions. It was found that under summer conditions, a limiting turbidity of 80 to 100 gave satisfactory conditions. Lately, under winter conditions, a limit of 50 has been in force. The limits are plainly lower than necessary, as reference to the turbidity curves will show.

In the absence of satisfactory data on this point from the records of the experimental work, analysis has been made of these turbidity curves for the outlet at Dalecarlia Reservoir and for the intake at the filtration plant for the past two years. During these, and only these years, the gates and by-passes of the reservoir system have been manipulated in a manner designed to give the best possible water supply to the filters. The study of these curves indicates limiting turbidities for coagulation of about 150 for summer and 100 for winter conditions, and these figures have been used in the estimates of quantity of coagulant required.

The quantity of coagulant necessary to properly treat different waters is shown in the accompanying diagram. The relations for the Potomac water under the conditions studied follow quite generally along the line corresponding to the schedule used by Mr. Allen Hazen for clay-bearing waters. For purposes of estimate, therefore, Mr. Hazen's schedule will be used.

The application of aluminum sulphate at the rate of 100 lb. of 17 per cent. sulphate per million gallons of water causes the decomposition of 6 parts per million of the dissolved carbonates of calcium and magnesium. From this definite relation the maximum allowable quantity of the coagulant for the most adverse conditions may be determined from the minimum quantity of dissolved carbonates ever found. This minimum quantity of carbonates in the Potomac water, from daily analyses covering a period of more than two years, is 31 parts per million. The maximum allowable quantity of aluminum sulphate, therefore, for this critical period of low carbonates is 516 lb. per million gallons. The highest turbidity that has reached the proposed location for the coagulating plant during the two years that the gates have been manipulated for the exclusion of high turbidities, was 500. The quantity of aluminum sulphate necessary for proper coagulation of this turbidity of 500 is 360 lb. per million gallons of water. In other words, the maximum allowable quantity of aluminum sulphate, under the most adverse conditions that have been observed, is 43 per cent. more than is necessary for the proper coagulation of the worst water occurring in two years of operation under present conditions. This seems a generous margin for possible error. The fact must be emphasized, too, that the above statement applies only to the most unfavorable combination of conditions that have been observed, and the probability of their frequent and simultaneous occurrence is very small. Every improvement in the conditions controlling this limit increases the margin for error on the side of safety, which for average conditions would be about .250 per cent.

Defects in Mechanical Principles Involved.—As heretofore stated, the primary object of these experiments was to determine whether any preliminary process involving purely mechanical means for the removal of the impurities in the water at times of high turbidity would be found as effective in that removal as coagulation and sedimentation. The results are set forth in this report. They demonstrate clearly the entire insufficiency of the mechanical processes at the very time when such a preparation of the water is most needed. They add convincing evidence to the mass of data that has resulted from the successful operation of many coagulating plants, showing the unfailing applicability of this method of treatment to the purpose herein stated.

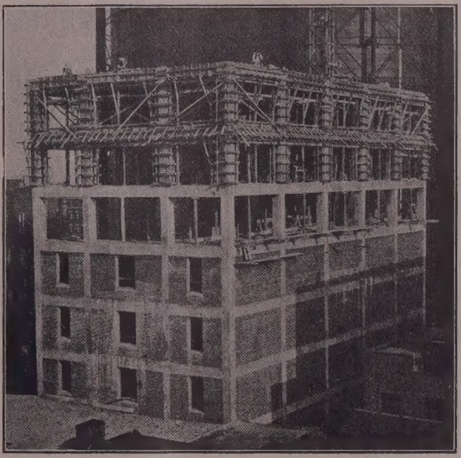
For the entirely successful treatment at all times of a clay-bearing water of which the Potomac is a type, the principles involved in Arrangements 1 and 2 are fundamentally wrong. This statement is true of every process that has come to attention in this investigation in which mechanical means alone are depended upon for the purification; and there is nothing to indicate that it is not true of every such process. The reasons upon which this statement is based are evident from a consideration of the following facts.

The impurities in water may be either in solution or suspension; in this case only the latter is considered. These may be classed generally as organic or inorganic. Besides sedimentation, there are three ways in which they may be removed: 1. Impurities of either class may be removed by a straining process, if they are greater in size than the openings in the medium interposed to arrest them. 2. If smaller than these openings, they may still be removed by being deposited upon the surfaces within the pores of this medium. This is merely a highly induced sedimentation process, and while remarkably efficient under favorable conditions, it has, from its very nature, certain limitations. 3. Some of the organic impurities, by bio-chemical changes, are broken down into more elementary forms, and are removed from this present consideration by going into solution.

The problem in the preliminary treatment of the Potomac water consists, not simply in the removal of a considerable percentage of the suspended matter at times of high turbidity, but essentially, the removal of those very particles which are not susceptible of removal by any of the three means above mentioned. This fine material is present in the Potomac water in some quantity at all times. At times of freshet when there is an increase in the total quantity of suspended matter in the water, there is a somewhat similar increase in the quantity of this material that is too fine to be removed under the given conditions. These particles are sub-microscopic in fineness; their diameters have been estimated at from 0.0001 to 0.0003 mm. The individual particles cannot be distinguished under the highest power microscope, under which the motions and structure of the bacteria may be plainly seen. In fact, there is room for conjecture as to whether this fine material exists as individual particles. There is reason for believing that it exists in that peculiar, unstable state of aggregation known as the colloidal state, which is on the border line between true solution and suspension. In brief, the turbidity which persists in the effluent of the filters is of almost inconceivable fineness.

In order to effect the removal of such matter as this from the water by the first means given above, the openings in the straining medium would have to be of equally inconceivable fineness. It is evident that gravel, coke, sponge and sand do not fulfil this requirement; and microscopical examination of other material employed, such as powdered coke, asbestos, and so forth, show that the openings in the straining layers they might form are relatively large for the arresting of this fine material. A medium to remove this clay by straining would have to be so fine that water would pass through it at a rate prohibitively low for application to a large municipal supply. Even such supposedly perfect devices as the Pasteur and the Berkefeld filters will not remove it all under all conditions.

To effect the removal of this material by the second means, it seems hard to conceive of anything more efficient than a bed of filter sand. Mr. Hazen has computed that the opportunities afforded for this induced sedimentation in a layer of filter sand of the conventional depth, are equivalent to those in a sedimentation basin with an area of more than 400 times that of the filter. Yet as the results of operation show, the sand filter will not remove it.


Building of the Bowling Green Storage & Van Co.

The defects in the principles, therefore, are apparent, and the general statement may be made that processes involving only these purely mechanical principles, for application to a clay-bearing water of the type under observation, are capable of removing a considerable percentage of suspended matter within certain limits, but are quite incapable of removing the very impurities at which the desired preliminary treatment is largely aimed.

Applicability of Coagulation Treatment.—Fortunately, in view of the failure of mechanical means for the successful preliminary treatment of this water, the success of another principle for application to this problem has again been demonstrated. That principle, which was involved in the operation of the No. 3 arrangement, is based on the simple increase in size of these clay particles until they are large enough to be visible to the naked eye, and to be easily and thoroughly removed by sedimentation and sand filtration. This is the effect produced by coagulation.

The result of this action is a mass of particles which, both chemically and mechanically, are quite stable. They are made up of aluminum hydrate, clay and other turbidity-producing material, bacteria, microscopical organisms, and perhaps other impurities from the water. They are large enough to be easily visible to the naked eye, and to be removed by a simple straining action. Their hydraulic value is great enough so that they may be removed in large part by sedimentation. They are suspended in the treated water, which otherwise is now crystal clear and much reduced in bacterial content. This water is now subjected to prolonged sedimentation, and then goes to the filters carrying but a little of its original burden of impurities, and these in a form to be easily and thoroughly removed.

The preliminary treatment of the Potomac water in the manner proposed should effect certain improvements in the quality of the filtered water, as indicated by the results of operation of Arrangement No. 3, reported herein. The turbidity, instead of being present in visible quantity during one-fifth or one-sixth of the time, should be eliminated so that there would never be any trace of it visible to the ordinary consumer.

The bacteria, which appear in the filtered water in unusually large numbers at time coincident with the high turbidities, should by the same means be reduced in number so that they would never under ordinary conditions much exceed the high standard set by the better of the averages shown in the table of bacterial results.

Resumé.—The results of eleven months of operation of the experimental plant have demonstrated the following facts: 1. The desired removal of turbidity and of bacteria from the Potomac water at the outlet of Dalecarlia reservoir cannot be effected by means of devices of the type of preliminary filters No. 1 and No. 2. The evidence tends to show, and there is every reason to believe, that this removal cannot be effected by any device applicable to large quantities of water, that involves purely mechanical principles in its operation. 3. The desired improvement in the water can be effected by occasional coagulation, with subsequent thorough sedimentation in the two existing reservoirs. This process is so entirely flexible that with its use the perfect final product of the filters so much desired is assured. 4. Consideration of the chemical results of coagulation, and the other pertinent facts presented, leads to the conclusion that the application of aluminum sulphate to the Potomac water, as proposed, does not present the slightest menace to the public health of the District of Columbia.

Rapid Concrete Building Construction.

The accompanying photograph of the building of the Bowling Green Storage & Van Co., on West 65th St., New York City, was taken on Nov. 13, 1907, and shows how closely it is possible to follow after the concreting of the skeleton framework of a reinforced concrete building with the curtain walls and incidental interior work. A close inspection of the photograph will show that the placing of the brickwork of the curtain wall has just been commenced in the two corner panels of the story from which the column forms have been stripped, while the concreting of the columns and floors is in progress in the second floor above.

The building in question is a seven story and basement structure, measuring 50 ft. by 100 ft. 5 in. The reinforced concrete skeleton was put up in 57 working days. The building bears a close resemblance to the McNulty Building, designed and erected by the same firm, and described in The Engineering Record of Dec. 14, 1907.

Tucker & Vinton, New York City, were the engineers and constructors.

TEN-FOOT VENTURI METERS with a 3 to 1 throat ratio have been installed in the Divi irrigation pumping plant, Madras Presidency, India, according to "The Engineer," London. The throat sections and pressure chambers are of wrought iron and the remainder of reinforced concrete.

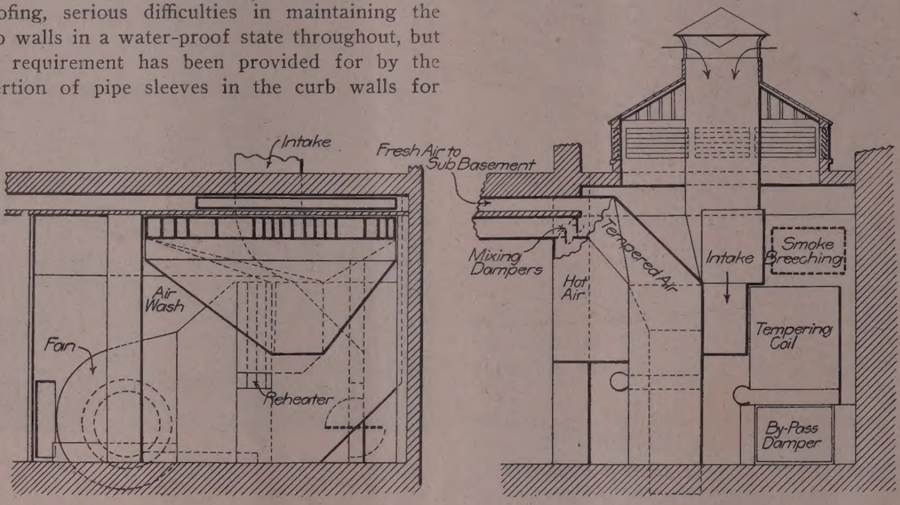
Mechanical Equipment of the Union National Bank Building, Pittsburg, Pa.

One of the most recent of the large business buildings erected in Pittsburg is a 21-story structure that has been built for the Union National Bank of that city, for use as banking offices on the first, mezzanine and basement floors, with rentable offices on the upper floors from the second to the twentieth inclusive. It is a steel-cage structure of modern fire-proof construction, and occupies a plot nearly square in shape, facing 84 ft. on Fourth Ave., and 86 ft. on Wood St. The sub-surface floors, comprising a basement and sub-basement, are extended under the sidewalks on both streets, enlarging the available space on these levels to an interior dimensions approximately 92 ft. square. The twenty-first floor is finished off as a roof house, 65 x 77 ft. in plan, and contains storage rooms, a kitchen and dining-rooms for the bank employees. Light and ventilation is afforded the rear portion of the upper stories by an 18 x 42-ft. light court, that rises from the second floor level, with an 18 x 21-ft. extension down to the basement level.

The mechanical services that are provided for in the building are particularly complete, embracing besides the usual heating, elevator and electrical services, an extensive ventilation system for the banking offices, refrigerating apparatus, vacuum-sweeping system, and so forth. The operation of these services necessitated an extensive mechanical plant, which has been installed with a boiler capacity of 800 hp. and requiring the entire sub-basement space. This sub-surface space was excavated for an unusual ceiling headroom of 21½ ft., with the floor level 33 ft. below level of curb in the street, in order to provide ample room for overhead piping connections. This space is undivided, except at the rear side, containing the electrical generators, pumping machinery, ventilating apparatus, and other equipment in one large room; the boilers are separated from the main room in a 35 x 50-ft. enclosure in the southwest corner, while adjoining the latter under the Wood St. sidewalk there is a 10 x 40-ft. enclosure that serves as a coal-storage vault. Above the vaults are two sidewalk coal holes through which the coal is dumped directly from coal wagons in the street, the vault having a capacity when filled of over 250 tons.

In the construction of the sub-surface space for the mechanical plant, difficulties were encountered with ground water, which stands normally at a high level above the lowest point of the excavation, the elevation of the curb at the corner of the two streets being but 39.8 ft. above the datum taken at the bottom of the Monongahela River adjoining, while the sub-basement floor level is but 6½ ft. above datum. The soil at the site is sand in the upper levels, with gravel underlying it, through which the water percolates from the river, as the latter is but a few blocks distant, so with the water at ordinary stages in the river the ground-water level stands close to the sub-basement ceiling. In winter freshets, the water has been known to rise within a very few feet of the curb level at this point, a stage of 33 ft. having been reached in the Monongahela River in the spring of 1907. It was accordingly necessary in order to prevent interference with the operation of the mechanical plant, to carefully water-proof the sub-surface walls, the foundations and the sub-basement floor, which has been provided for with tile drains underneath the latter for the removal of seepage to a sump tank. The water-proofing as applied is a lining of Hydrolithic cement, which is applied in two coats to a thickness of about ⅝ in. over the inside surface of the side walls at both basement and sub-basement levels, the coatings being continuous on these surfaces, except where the basement floor beams enter the curb walls. At the sub-basement floor level, the lining is carried down in a continuous sheet into and through the lower portion of the floor slab. The drainage course under this floor construction consists of 8 in. of broken stone, through which are laid lines of 4-in. tile drain, the broken stone resting on a thin course of cinder concrete that has 6-in. holes at 2-ft. intervals, to permit of infiltration of the ground water. The water-proofing is said to have given very satisfactory results, no water having entered the basement during the flood of March, 1907, except through the breaks in the water-proofing course in the curb walls around the basement floor beams, and even in that emergency no difficulty was experienced in relieving the pressure on the walls by removing the water from the sump tank as fast as it could flow in, which was accomplished by two pumps in the mechanical plant, having each a capacity of 300,000 gal. per day.

The entrance of sewer, water and other piping connections from the street into the building through the sub-surface walls, involved, with these conditions of ground water and water-proofing, serious difficulties in maintaining the curb walls in a water-proof state throughout, but this requirement has been provided for by the insertion of pipe sleeves in the curb walls for the various connections, there being in all 13 sleeves installed, varying from 2 in. to 12 in. pipe, for city water, sewer connections, gas and other piping connections and also for telephone, telegraph, messenger and other cable connections. The sleeves are all of heavy galvanized wrought-iron pipe, with flanges screwed on either end. To the flange at the outer side of the wall is bolted a companion flange to retain a ring of packing around the outer surface of the sleeve and to insure tightness at the outer end, while to the inner flange is bolted a galvanized blank flange which is drilled and tapped to screw over the entering pipe. The latter is screwed into place in the sleeve from outside of the wall and then caulked tightly within the sleeve by asphalt and oakum for a water-tight connection. The piping connections within the basement are then made directly from the end of the pipe which extends through the inside cap flange.



Details of Supply Fan, Heaters and Air Wash Apparatus.

A feature of the mechanical equipment of the building is the ventilation system which has been provided for the banking offices on the first and mezzanine floors and in the basement, and also for the machinery spaces in the sub-basement. Both fresh air supply and exhaust ventilation have been provided for in all rooms on these lower floor levels; the fresh air delivered is passed through an air-wash apparatus for cleaning it of the dust and soot characteristic of the Pittsburg atmosphere. The fresh-air supply has been made practically independent of heating in the portions of the building served, it being the intention to merely raise the air to the desired interior temperature for delivery, while heating is provided for by direct radiation; the arrangement is such, however, that partial heating may be accomplished by the indirect system if desired. The ventilation is applied to the rooms in general on the downward system, the fresh air entering the rooms at elevated points, while the exhaust openings are located near the floor lines. A favorable location for the fresh-air supply fan was permitted by the intake connection at low level in the interior court as made possible by the use of the air-wash system, the fan for this service being located underneath louvres at the base of the light court where most convenient for the connections for the duct risers to the upper floor levels. The exhaust ventilation is operated by fans, located at the roof level which draw through vent flues.

The fresh-air supply equipment consists of a 6 x 3½-ft. steel plate contrifugal fan, which is located in an enclosure in the sub-basement underneath the 18 x 20-ft. extension of the interior lighting court down to basement level, and delivers through tempering coils, the air-washing equipment, reheating coils and mixing dampers to the distribution duct-work system. The fan is a three-quarter-housed blower, with top horizontal discharge casing, and is direct-connected to a 6 x 9-in. horizontal simple steam engine, both having been built by the American Blower Co. The unit is designed to operate at 230 r.p.m, at which speed the fan has a delivery capacity of 26,300 cu. ft. per minute. This fan outfit is located, together with its two sets of heating coils and the air-washing apparatus, in a small space 20 x 25 ft. in extent between building columns and partitions, an arrangement having been developed as shown in the accompanying drawings of the fan room. The intake is through an air inlet monitor in the interior court above, which has a rectangular duct connection, 4½ x 5 ft. in cross sectional area, leading down to a cold-air chamber in the rear section of the fan-room. This chamber contains a small tempering coil for preheating air before it enters the spray chamber of the air-washing apparatus beyond, which consists of the usual arrangement of a spray chamber and baffle plates in the drying chamber, from the latter of which the air enters the fan. The fan has two delivery outlets, one a tempered air chamber and the other through a bank of reheater coils for the final increase in temperature, from each of which divisions there are connections in duplicate to each fresh-air delivery duct with mixing dampers for accurate proportionment of tempered and reheated air, and consequent adjustment of delivery temperatures. The arrangement of the apparatus is, as shown in the drawings, in two rows, with the air-washing apparatus at the rear and the fan with its engine and the ducts and mixing dampers, to the front where most accessible. Considerable economy of